CONCRETE MASONRY

foundation walls

NATIONAL CONCRETE MASONRY ASSOCIATION • 38 SOUTH DEARBORN • CHICAGO

The Function of the National Concrete Masonry Association, a non-profit promotional, educational and technical organization, is to improve and extend the uses of concrete masonry through scientific research and development of new or improved products and methods. The NCMA is comprised of leading manufacturers of quality masonry units in the United States, Canada, Hawaii, Alaska, Puerto Rico, Japan, Venezuela and New Zealand. A complete list of members will be sent on request.

DESIGN AND CONSTRUCTION OF PLAIN AND REINFORCED TYPES

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TABLE OF CONTENTS

Page

I. INTRODUCTION 1

II. GENERAL DESIGN CONSIDERATIONS AND ASSUMPTIONS. 2

III. LOADS ON FOUNDATION WALLS

A. Dead Loads. 3
B. Live Loads 3

1. Lateral Earth Pressure 5
2. Wind and Blast Forces 6
3. Earthquake Forces 7

C. Moments and Shears 9

IV. PLAIN CONCRETE MASONRY FOUNDATION WALLS

A. Design Tables and Recommendations 13

1. Materials 13
2. Allowable Stresses 14
3. Recommended Minimum Wall Thickness 19
4. Lateral Support 20
5. Pilasters. 22
6. Bond Beams and Joint Reinforcement 25
7. Expansion and Control Joints 28
8. Pier Foundations 29

B. Construction Details and Recommendations. 31

1. Footings 31
2. Wall Construction 33
3. Anchorage 34

V. REINFORCED CONCRETE MASONRY FOUNDATION WALLS

A. Design Tables and Recommendations 36

1. Minimum Reinforcement Requirements 42
2. Allowable Stresses 43
3. Minimum Wall Thickness and Lateral Support 45
4. Effective Sections for Stress Calculations 45
5. Combined Bending and Direct Stress 46
6. Deflections. 46

Page

B. Recommended Construction Details and Practices 47

1. Footings 47
2. Wall Construction 47

a. Units 47
b. Laying Up the Walls 47
c. Reinforcement 52
d. Mortar 52
e. Corefill 52
f. Placing Corefill 53

3. Anchorage 53
4. Openings and Miscellaneous Details 54

VI. WATERPROOFING, CONDENSATION, AND TERMITE PROTECTION

A. Waterproofing 56

1. Surface and Subsurface Drainage 56
2. Waterproof Coatings 57
3. Foundation Walls 57
4. Basement Walls 58

B. Condensation 58
C. Termite Protection 59

VII. ERECTION PRECAUTIONS 62

VIII. SUGGESTED SPECIFICATIONS 63

IX. SELECTED REFERENCES 65

APPENDIX A. Provisions of the American Standard Building Code Requirements for Masonry . . . 67

APPENDIX B. Typical Reinforced Concrete Masonry Basement Wall Calculations

I. Nomenclature 71

II. Typical Calculations 73

APPENDIX C. Illustrations of Typical Concrete Masonry Foundation Wall Construction

I. Plain Concrete Masonry Construction 80

II. Reinforced Concrete Masonry Construction 84

LIST OF TABLES

Table		Page
1	ASTM Physical Requirements for Grade A Hollow and Solid Load-Bearing Concrete Masonry Units and Concrete Brick	13
2	Mortar Requirements	14
3	Allowable Compressive Stresses and Axial Compressive Loads on Unreinforced Concrete Masonry Foundation and Basement Walls	15
4	Allowable Stresses in Flexural Tension and Shear in Unreinforced Concrete Masonry, - Uniform Building Code, 1955 Edition	16
5	Flexural Strength of Concrete Masonry Walls in the Vertical Span	16
6	Flexural Strength of Concrete Masonry Walls in the Horizontal Span	17
7	Racking or Horizontal Shearing Resistance of Concrete Masonry Walls	17
8	Tensile Bond Strength Developed Between Mortar and Concrete Masonry Units as Determined in Eccentric Load Tests of Two-Block Piers	18
9	Shear Bond Strength Developed Between Mortar and Concrete Masonry Units as Determined in Transverse Load Tests of Three-Block Piers	19
10	Recommended Maximum Wall Heights and Depths Below Grade for Unreinforced Concrete Masonry Basement Walls	21
11	Allowable Lateral Flexural and Shearing Resistance of Reinforced Concrete Masonry Bond Beams	26
12	Calculated Ultimate Resisting Moments in the Horizontal Span for Concrete Masonry Walls Reinforced with Joint Reinforcement	28
13	Allowable Bearing Value of Soils	33
14	Recommended Vertical Reinforcement for Reinforced Concrete Masonry Basement Walls for Various Wall Heights and Depths below Grade	38
15	Recommended Vertical Reinforcement for Partially Reinforced Concrete Masonry Basement Walls for Various Wall Heights and Depths below Grade	40
16	Recommended Allowable Stresses in Reinforced Concrete Masonry	44
17	Ultimate Compressive Strength of Two and Five-Block High Hollow Concrete Masonry Unit Piers	44

LIST OF FIGURES

Figure		Page
1	Typical Earth Pressure Diagrams and Formulas Used in Calculating Moments and Shears in Foundation Walls Resisting Earth Pressure Loadings through Bending in the Vertical Span .	4
2	Effect of Superimposed Loads in Increasing Lateral Earth Pressure on Basement Walls. . .	5
3	Effect of Wind in Producing Loads on Buildings and Building Foundations	6
4	Effect of Translation of the Superstructure in Causing Bending in Foundation Walls Extending Perpendicular to the Direction of Movement	6
5	Effect of Earthquake Movements on Buildings and Building Foundations.	7
6	Rotational Effects of Earthquake Forces on Foundations.	8
7	Moment Coefficients for Median Vertical and Horizontal Strips in Wall Hinged at Top and Bottom and Fixed at Ends	9
8	Bending Moments and Shears in Foundation Walls Resisting Earth Pressure through Bending in the Vertical Span - Type A Soil	10
9	Bending Moments and Shears in Foundation Walls Resisting Earth Pressure through Bending in the Vertical Span - Type B Soil	11
10	Approximate Distribution of Earth Pressure Load to Vertical and Horizontal Spans of Wall Hinged at Top and Bottom Edges and Fixed at Ends	12
11	Assumed Distribution of Concentrated Loads in Walls Where Units are Laid in Common or Running Bond	15
12	Typical Hollow Unit Concrete Masonry Wall Sections	20
13	Effect of Axial Compressive Loads in Increasing the Bending Resistance of Hollow Unit Concrete Masonry Walls in the Vertical Span	20
14	Effect of Crosswalls and Offsets in Reducing Wall Spans in Horizontal Bending.	21
15	Effect of Backfilling before Top Support is in Place on Moment and Shear in Foundation Wall .	22
16	Typical Concrete Masonry Pilaster Designs	23
17	Approximate Analysis of Pilaster Serving as a Lateral Support for Basement Walls	24
18	Approximate Analysis of Pilaster Serving to Stiffen Basement Walls in the Vertical Span . .	24
19	Types and Suggested Location of Bond Beams in Concrete Masonry Foundation Walls	25

Figure

Page

20 Types and Suggested Location of Horizontal Joint Reinforcement in Concrete Masonry Foundation Walls 27

21 Horizontal Section at Mortar Joint Containing Embedded Joint Reinforcement 27

22 Typical Shear-Resisting Control Joint Details 29

23 Pier Foundations 29

24 Analysis of a Pier Foundation for a Small Building. 30

25 Typical Reinforced Pier Sections 31

26 Recommended Dimensions for Footings Supporting Plain Concrete Masonry Foundation Walls. 31

27 Typical Corner Construction for 8, 10, 12, and 16 in. Walls 32

28 Typical Methods of Constructing Course Supporting Floors in Concrete Masonry Foundation Walls 33

29 Suggested Methods of Anchoring Wood Joists Bearing on Concrete Masonry Foundation Walls. 34

30 Typical Detail for Anchorage of Concrete Masonry Walls to Parallel Wood Joists or Beams . 34

31 Suggested Method of Bonding Bearing Walls at Intersections. 35

32 Recommended Vertical Reinforcement for 8 in. Reinforced Concrete Masonry Basement Walls 37

33 Recommended Vertical Reinforcement for 10 in. Reinforced Concrete Masonry Basement Walls 37

34 Recommended Vertical Reinforcement for 12 in. Reinforced Concrete Masonry Basement Walls 37

35 Recommended Vertical Reinforcement for 10 in. Partially Reinforced Concrete Masonry Basement Walls 42

36 Recommended Vertical Reinforcement for 12 in. Partially Reinforced Concrete Masonry Basement Walls 42

37 Principal Details of Piers Tested for Ratio, f'_p/f'_b, and Ratio, E_p/E_b 45

38 Effective Sections Assumed for Stress Calculations in the 8 in. Reinforced Concrete Masonry Basement Wall Designs 45

39 Suggested Construction Details for Reinforced Concrete Masonry Foundation Walls 48

40 Suggested Construction Details for Reinforced Concrete Masonry Foundation Walls 49

41 Typical Dimensions of Three and Two Core Concave-End Units and Wall Constructions. . . 50

42 Typical Dimensions of Two Core Plain-End and Open-End Units and Wall Constructions . . 51

43 Typical Methods of Placing Horizontal Bond Beam Reinforcement at Corners. 52

44 Typical Detail of Tie to Hold Vertical Reinforcement in Place While Cell is Filled 52

45 Typical Foundation Wall - Floor Framing and Connection Details 53

Figure Page

46 Typical Foundation Wall - Footing Connection Where Basement Slab is Not Designed to Resist Inward Wall Reaction 54

47 Typical Reinforcing and Window Detail at Openings in Reinforced Concrete Masonry Basement Walls 54

48 Suggested Methods of Waterproofing Concrete Masonry Basement Walls 58

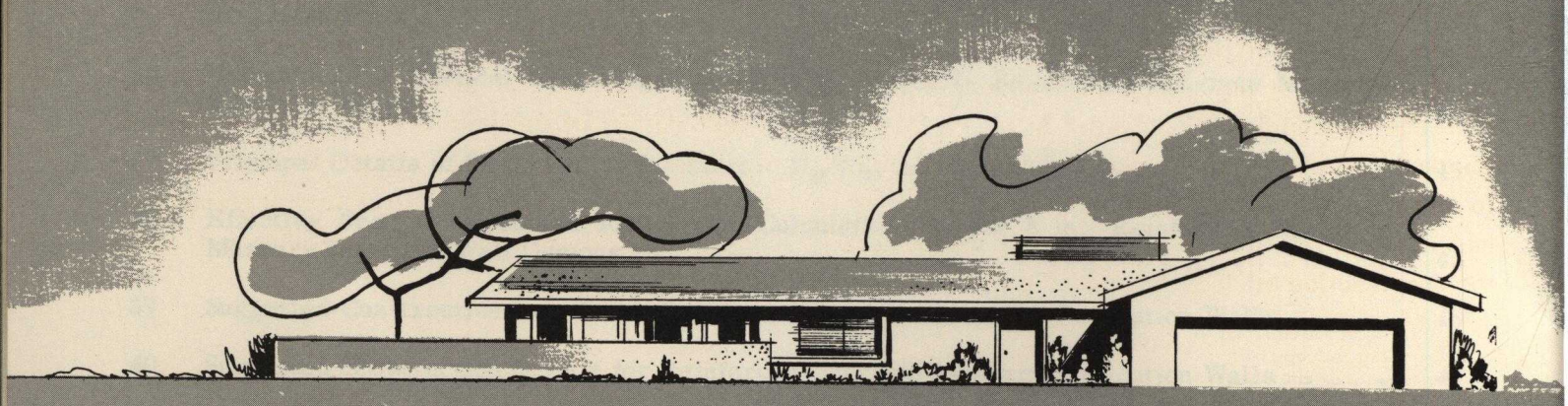
49 Comparison of Insulating Value of 8 in. Poured Concrete and 8 in. Concrete Masonry Basement Walls 59

50 Geographic Distribution of Termites 59

51 Recommended Methods of Capping Concrete Masonry Foundations Supporting Wood Frame Construction in Termite Infested Areas 60

52 Recommended Methods of Capping Concrete Masonry Foundations Supporting Masonry Exterior Walls in Termite Infested Areas 60

53 Termite Shield Details 61



I. INTRODUCTION

Foundation walls are building walls below the floor nearest grade serving as a support for walls or other structural members, as enclosure walls around excavated areas or as both supporting and enclosure walls. Foundation walls enclosing usable areas under a building are also called basement walls. Due to its excellent durability, compressive strength, and resistance to fire, termites, rodents and other destructive forces, as well as its economy and availability, concrete masonry is well suited for use in foundation wall construction.

In the past, use of concrete masonry in foundations has been confined almost wholly to residential and light commercial construction where vertical and lateral loads carried by the foundations seldom exceed those which can be safely resisted by plain concrete masonry of practical and economical thickness. In recent years, the development of reinforcing techniques similar to those used in conventional reinforced concrete has made it possible to use concrete masonry in the construction of deep basement walls, foundation walls supporting heavy compressive loads, and basement walls generally where the unsupported wall length or height will exceed that recommended for plain concrete masonry.

The discussion, design tables and suggested construction details presented in the following have been prepared primarily to assist architects and engineers in the design of both plain and reinforced concrete masonry foundation and basement walls, and to acquaint builders and contractors with recommended construction details and practices. Since soil conditions, building code requirements and other factors affecting the design may vary appreciably even in the same locality, this material is necessarily of a general nature and intended only to be helpful in illustrating typical design conditions, assumptions, and procedures. The drawings included are not intended to serve and should not be used as working drawings. The actual design of foundation walls, preparation of working drawings, and similar services are best rendered by a qualified architect or structural engineer familiar with local conditions and code requirements.

II. GENERAL DESIGN CONSIDERATIONS AND ASSUMPTIONS

The principal factors to be considered in the design of foundation walls are: 1) the vertical and horizontal loads which the walls must resist; 2) the manner in which the walls are intended to carry and transmit the loads; and 3) the general structural design of the building. Both the load applied to the walls and the wall design will vary with the general building design. For example, basement walls supporting bearing wall construction usually are required to support comparatively heavy compressive loads in addition to earth pressure or other lateral loads and to span appreciable distances horizontally between intersecting cross partitions or building corners. Such walls may be designed to transmit lateral loads to the floors and footings through bending in the vertical span, and little or no horizontal distribution of the lateral load assumed. In skeleton frame construction, wall columns may extend down to separate footings and carry most of the superstructure dead and live loads, in which case basement walls may be subject to appreciable lateral but little vertical load. If the columns are closely spaced, the walls may be designed to transmit lateral loads horizontally, or both horizontally and vertically--as two-way slabs. If the vertical supports are widely spaced and the first floor construction cannot be considered as providing lateral support, a design assuming cantilever action usually will be required. *

The data and tables relating to the design of reinforced concrete masonry foundation walls included in this booklet apply principally to walls designed to carry bending and shearing loads as vertical one-way beams or slabs, simply supported at the top and bottom. Although the assumption that lateral loads are carried entirely by the vertical span is not theoretically correct, since some portion will be carried through bending in the horizontal direction, it usually is justified whenever the wall length between vertical lateral supports exceeds three times the wall height. It has the advantage of simplifying the design problem and giving conservative results. When the ratio of wall length to wall height between lateral supports is under about two, it may be advisable and more economical to employ a more exact design assuming both horizontal and vertical load distribution.

Designers interested in the design of foundation walls by more exact methods, as briefly discussed on page 12, should refer to available published literature on the design of walls as two-way slabs or as thin plates supported on two, three or four sides.

* For information on the design of cantilever type concrete masonry foundation walls see "Reinforced Concrete Masonry Retaining Walls," published by the National Concrete Masonry Association.

III. LOADS ON FOUNDATION WALLS

Loads carried by foundation walls may be divided into dead loads, those due to the weight of the superstructure and the foundation itself, and live loads, those resulting from application of other forces or pressures to the foundation wall or the superstructure supported thereon. Loads may be further classified as either vertical or lateral, as static or dynamic, or as concentrated, uniformly distributed, or other, depending upon their source and nature and the manner in which they are transmitted to the foundation. Since a safe and economical foundation wall will be one which will carry the maximum expected loads with a reasonable factor of safety, it is evident that the problem is greatly dependent upon the assumptions made as to the loads acting upon the superstructure and the foundation.

A complete discussion of the various forces which require consideration in building design is beyond the scope of this information but it is advisable to briefly review the subject as it pertains more especially to the design of foundations.

A. DEAD LOADS

The dead load acting on a foundation wall will consist of the weight of the superstructure bearing on the wall and of the wall itself, and usually may be quite easily calculated from the weights of the materials used in the construction. With some exceptions, the dead load may be assumed to be uniformly distributed over the wall length and thickness. It should be remembered, however, that the reactions of flexural members bearing on foundation walls, such as floor slabs, beams, girders and joists, ordinarily will fall somewhere between the wall centerline and the inner wall surface rather than along the axis of the wall, and that the eccentricity of such loads will tend to induce some bending moment and increase the maximum compressive stresses. Also, where flexural members are built into or otherwise restrained at the foundation wall, moment will be induced in the wall by virtue of the continuity developed at the connection. Generally, such induced moments will tend to counteract those due to lateral earth pressure acting on the exterior wall face so that it will be conservative, in most cases, to neglect them in the design. With heavy loads and appreciable eccentricities, however, or when moments induced by continuity or eccentric vertical loads act with rather than against moments due to lateral loads, designers should investigate and include their effect in the design calculations.

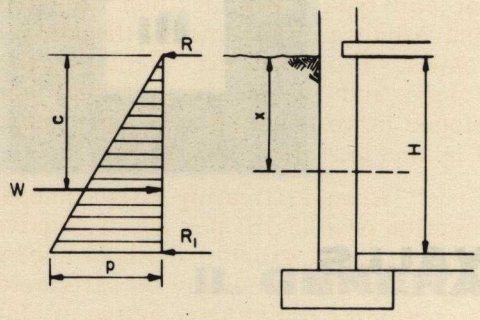
In the design of foundation walls, it is usually assumed that vertical compressive stresses resulting from dead load will be effective in reducing any vertical tensile stresses developed in resisting bending or direct loads. In this regard, it is important to remember that only moments and stresses due to dead load may safely be considered as opposing moments and stresses from lateral forces. Also, precautions should be taken in the erection to insure that the dead load counted upon in the design will actually be present and acting on the foundation before the foundation is subjected to lateral load. For example, if the design of a basement wall is based on the assumption that the vertical dead load of the superstructure will act to reduce the tensile stresses due to earth pressure moment, the walls should not be backfilled prior to erection of the superstructure. If early backfilling is unavoidable, temporary bracing should be provided as required to insure that the wall stresses will not exceed the working stresses assumed in the design.

B. LIVE LOADS

Live loads consist of any load applied to a structure or structural element other than the weight of the structure or element itself. They may also be defined as variable, transient, or moving loads: as for example, the weight of snow on a roof; the weight of persons, movable objects, or stores on a floor; lateral earth pressure on a basement wall; or loads due to dynamic forces such as wind, blast, earthquake and impact forces.

The type and magnitude of live load to be considered will vary according to locality,

CASE I: Finish grade level with bottom of first floor. No surcharge.



$p = \omega H$, where ω = equivalent-fluid weight of soil

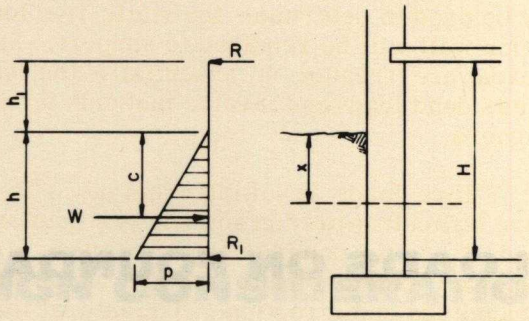
$$W = \frac{\omega H^2}{2} = \frac{pH}{2}\ ;\quad c = \frac{2H}{3}\ ;\quad R = \frac{W}{3}\ ;\quad R_1 = \frac{2W}{3}$$

Maximum Shear: $V = R_1 = \frac{2W}{3}$

Point of Maximum Moment: $x = \frac{H}{\sqrt{3}} = 0.5774\,H$

Maximum Moment: $M = \frac{2WH}{9\sqrt{3}} = 0.1283\,WH$

CASE II: Finish grade below bottom of floor. No surcharge



$p = \omega h$, where ω = equivalent-fluid weight of soil

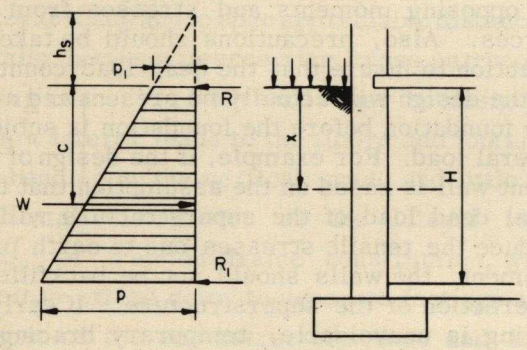
$$W = \frac{\omega h^2}{2} = \frac{ph}{2}\ ;\quad c = \frac{2h}{3}\ ;\quad R = \frac{Wh}{3H}\ ;\quad R_1 = W - \frac{Wh}{3H}$$

Maximum Shear: $V = R_1 = W - \frac{Wh}{3H}$

Point of Maximum Moment: $x = h\sqrt{\frac{h}{3H}}$

Maximum Moment: $M = \frac{Wh}{3H}\left[h_1 + \frac{2h}{3}\sqrt{\frac{h}{3H}}\right]$

CASE III: Finish grade above bottom of first floor or level with bottom of first floor with surcharge.



$p_1 = \omega h_s$; where ω = equivalent-fluid weight of soil

$$h_s = \frac{\text{surcharge (lb./sq.ft.)}}{\text{weight of soil (lb./cu.ft.)}}$$

$$p = \omega(H + h_s)$$

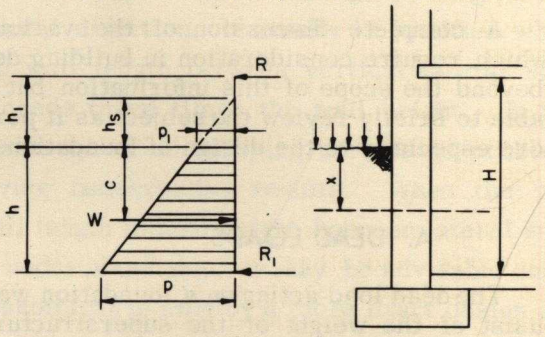
$$W = \frac{H(p+p_1)}{2}\ ;\quad c = \frac{H(2p+p_1)}{3(p+p_1)}\ ;\quad R = \frac{H}{6}(p+2p_1)\ ;\quad R_1 = \frac{H}{6}(2p+p_1)$$

Maximum Shear: $V = R_1 = \frac{H}{6}(2p+p_1)$

Point of Maximum Moment: solve for x in equation: $\frac{(p-p_1)}{H}x^2 + 2p_1x - 2R = 0$

Maximum Moment: $M = \frac{x}{6H}\left[H^2(p+2p_1) - 3Hp_1x - x^2(p-p_1)\right]$

CASE IV: Finish grade below bottom of first floor with surcharge.



$p_1 = \omega h_s$; where ω = equivalent-fluid weight of soil

$$h_s = \frac{\text{surcharge (lb./sq.ft.)}}{\text{weight of soil (lb./cu.ft.)}}$$

$$p = \omega(h + h_s)$$

$$W = \frac{h(p+p_1)}{2}\ ;\quad c = \frac{h(2p+p_1)}{3(p+p_1)}\ ;\quad R = \frac{h^2(p+2p_1)}{6H}\ ;\quad R_1 = W - R$$

Maximum Shear: $V = R_1 = W - R$

Point of Maximum Moment: solve for x in equation: $\frac{(p-p_1)x^2}{h} + 2p_1x - 2R = 0$

Maximum Moment: $M = \frac{h^2}{6H}(p+2p_1)(h_1+x) - \frac{x^2}{6h}\left[3hp_1 + x(p-p_1)\right]$

Figure 1 - Typical Earth Pressure Diagrams and Formulas Used in Calculating Moments and Shears in Foundation Walls Resisting Earth Pressure Loadings through Bending in the Vertical Span

occupancy, and other factors. Static live loads generally may be predicted to a reasonable degree of accuracy. Most building ordinances specify minimum design static live loads which will govern or may be used to determine the static live load a foundation wall will be required to support. Static live loads are treated in essentially the same manner as dead loads and involve methods familiar to designers.

Live loads resulting from other forces are more difficult to predict and analyze than static live loads due to the complex nature of these forces and the many indeterminate factors which affect their magnitude and direction. The lateral forces most frequently requiring consideration in foundation wall design are earth pressure, wind and blast, and earthquake.

1. Lateral Earth Pressure

Foundation walls enclosing excavated areas must resist a lateral force corresponding to the pressure of the retained earth against the exterior wall face. If it were permissible for the walls to yield sufficiently at the top to develop the full shearing resistance of the soil, the lateral force would be a function of the active soil pressure as usually assumed in the design of cantilever retaining walls. However, foundation walls of buildings are purposely restrained against horizontal yielding and are probably more subject to passive earth pressures, that is the lateral pressure of earth at rest, which may exceed the active value. Little information is available on this subject or on methods of estimating passive earth pressures. Accordingly, the accepted practice of using conservative active pressure values has been employed in this booklet.

Since the physical properties of the soil, soil conditions, and other factors influencing the pressure of retained earth on a wall may vary considerably even in the same locality, the magnitude of the earth pressure expected to act on a foundation wall can be accurately estimated only by someone familiar with actual conditions and with the type of soil in question. In large important projects where soil tests and foundation studies are made, theoretical methods of estimating lateral earth pressures may and should be used. When the soil properties are not determined by actual test, empirical or semi-empirical methods are generally employed. When estimating lateral earth pressures by empirical methods, it is advisable to check the design through comparison with previous construction of the same type and in the same area.

One simple, widely used, semi-empirical method of estimating lateral earth pressures is the equivalent-fluid method, in which it is assumed that the retained earth acts as a fluid in exerting pressure on a wall. Allowance is made for variations in soil type through assumption of a different equivalent-fluid weight for each appreciably different soil type. Fig. 1 illustrates typical earth pressure diagrams assumed in the design of foundation walls. Formulas for calculating the total lateral load and the maximum shear and moment developed in walls acting as simply supported vertical beams are also given in Fig. 1.

When estimating earth pressures, the assumption of a 25 lb. per cu. ft. equivalent-fluid for well-drained, granular soils of average or high permeability will generally be satisfactory. With soils containing a high percentage of clay, fine silt or similar materials and generally of low permeability, or with poorly-drained soils, a higher equivalent-fluid weight is recommended; 40 lb. per cu. ft. being used in this booklet. With very poor soils or soil conditions, as in the case of highly plastic clays and silts, actual soil pressures are difficult to estimate accurately by any method, and where such soils are encountered the services of a competent soils engineer should be employed.

Under certain conditions earth pressures on basement walls may be substantially increased through the application of superimposed loads on the earth adjacent to or near the walls. This may occur where a roadway, rail siding, stacked materials, or heavy equipment are located or operated near the structure during or subsequent to the construction operations. In allowing for the effect of such superimposed loads, designers frequently follow the practice of converting the superimposed load to an equivalent depth of fill. For example, a load of 300 lb. per sq. ft. on the earth adjacent to a basement wall would be considered as equivalent to 3 ft. of earth fill weighing 100 lb. per cu. ft. The magnitude and position of the resultant of the lateral earth pressure on the wall would then be calculated assuming a trazezoidal earth pressure diagram, as shown in Fig. 2, in lieu of the triangular diagram generally assumed where no superimposed load or surcharge need be considered.

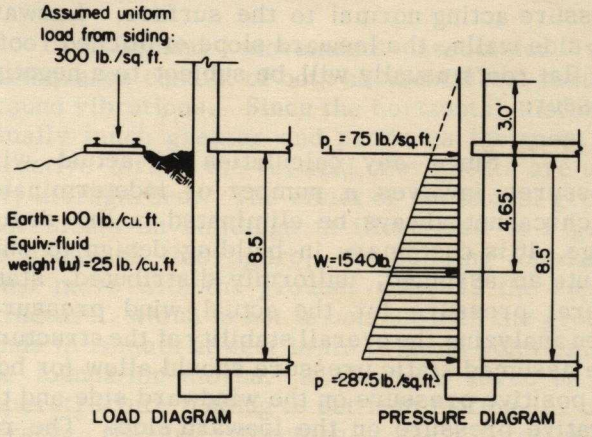


Figure 2 - Effect of Superimposed Loads in Increasing Lateral Earth Pressure on Basement Walls

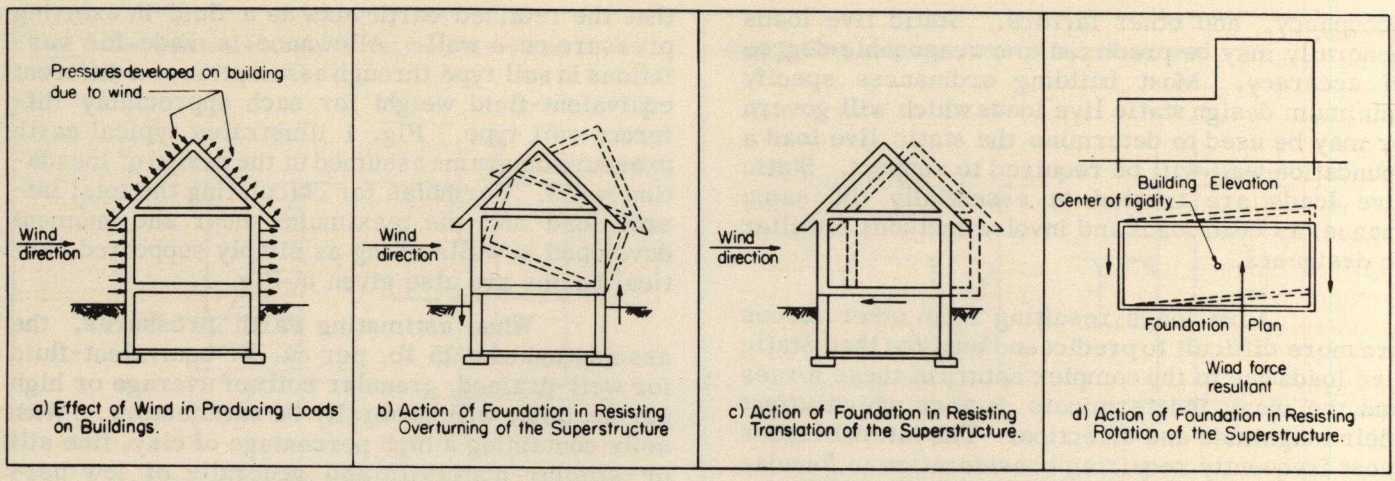

Pressures developed on building due to wind
Wind direction
a) Effect of Wind in Producing Loads on Buildings.
Wind direction
b) Action of Foundation in Resisting Overturning of the Superstructure
Wind direction
c) Action of Foundation in Resisting Translation of the Superstructure.
Building Elevation
Center of rigidity
Foundation Plan
Wind force resultant
d) Action of Foundation in Resisting Rotation of the Superstructure.

Figure 3 - Effect of Wind in Producing Loads on Buildings and Building Foundations

Where foundation walls are subject to lateral loads and transmit all or portions of such loads to columns, floors, or other structural members through beam action, the thrust of the walls on these members must be considered in their design.

2. Wind and Blast Forces.

Direct wind pressures on foundation walls seldom need be considered. The action of the superstructure in resisting wind pressures, however, will produce compressive, uplift, and shearing or racking loads on foundation walls which should not be overlooked in their design.

In calculating wind loads on foundation walls, it usually is assumed that the superstructure will act as a unit in resisting wind pressures and in transmitting the wind load to the foundation. Actual wind pressures will, of course, vary with the wind velocity, atmospheric conditions, size, shape, and orientation of the building, height above ground, amount of shielding and other factors. As shown in Fig. 3(a), windward walls and slopes of pitched roofs normally will be subject to a positive pressure acting normal to the surface. Leeward and side walls, the leeward slope of pitched roofs, and flat roofs usually will be subject to a negative pressure.

Since any calculation of actual wind pressures involves a number of indeterminates which cannot always be eliminated in the design stage, it is customary in building design to substitute an assumed, uniformly distributed, static lateral pressure for the actual wind pressures when analyzing the overall stability of the structure. The assumed static pressure should allow for both the positive pressure on the windward side and the negative pressure on the leeward side. The resultant of this assumed pressure uniformly distributed over the gross area of the vertical projection of the building elevation perpendicular to the wind flow is taken as the design wind load. Most buildings, not exceeding about 50 ft. in height, are designed to resist a lateral pressure of 15 to 20 lb. per sq. ft. acting in any direction. Higher design pressures are used where extreme wind velocities may be expected or where the building height will exceed 40 to 50 ft.

As shown in Fig. 3(b), (c) and (d), wind forces tend to cause overturning, translation, and/or rotation of the superstructure with respect to the foundation. In the analysis of the loads developed on foundation walls in resisting such movements, it is usually assumed that the overturning moment will be resisted by the foundation walls perpendicular to the direction of wind flow. This results in increasing the direct vertical compressive load on the leeward walls and decreasing that on the windward walls. It is further assumed that the force tending to cause translation will be

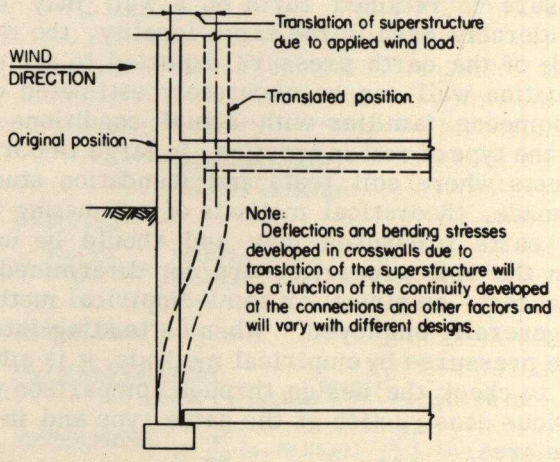

Translation of superstructure due to applied wind load.
WIND DIRECTION
Translated position.
Original position
Note:
Deflections and bending stresses developed in crosswalls due to translation of the superstructure will be a function of the continuity developed at the connections and other factors and will vary with different designs.

Figure 4 - Effect of Translation of the Superstructure in Causing Bending in Foundation Walls Extending Perpendicular to the Direction of Movement

transmitted to the foundation walls parallel to the wind flow by the first floor acting as a diaphragm. This will produce racking or horizontal shearing loads on these walls. Where the center of resistance to horizontal translation does not coincide with the line of action of the resultant wind force, a rotational moment will be developed which will affect the distribution of the racking load on the walls. Theoretically, translation of the superstructure also produces bending in the walls perpendicular to the movement, as illustrated in Fig. 4; however, the resulting stresses rarely are significant and usually are not computed.

Stresses developed in resisting overturning seldom will be critical except in foundations supporting light superstructures subject to high wind pressures. In such cases, the overturning moment may exceed the dead load moment and result in tensile or uplift loads on the windward walls. Accordingly, special attention should be given to the connections anchoring the superstructure to the foundation and to the foundation design itself to insure adequate strength in tension. Similarly, high horizontal shearing stresses may be developed in foundations supporting superstructures with a high ratio of exposed area to depth in the direction of wind flow. In such designs, special attention should be given to the shearing strength of the connections anchoring the superstructure to the foundation and the horizontal shearing resistance or racking modulus of the walls parallel to the wind flow. Where the floor bearing on the foundation will not have sufficient rigidity to permit the assumption of diaphragm action, high shear and bending stresses may be expected in the walls normal to the lateral force. In such cases, consideration should be given to the need for shear walls, pilasters, or bond beams to stiffen such walls.

Blast forces are quite similar to wind forces in their action on structures in many respects, since both are aerodynamic in nature. Their principal difference lies in the magnitude and the time duration of the pressures imposed on buildings; also their surcharge effect on earth pressures acting directly on foundations. Ordinary high explosives (non-nuclear) can create pressures exceeding normal wind pressures by many times in the case of buildings located near the blast source. However, the duration of time during which the maximum pressure acts is so short, usually measured in fractions of a second, that the pressure greatly dissipates before the building can respond or be stressed to failure. For this reason, it is usually permissible to assume a static lateral pressure equal to the average pressure which might be expected at the assumed distance from the blast force, rather than the maximum pressure. A design static pressure for non-nuclear explosions, ranging upward from 150 lb. per sq. ft., is used in some cases depending upon the importance and location of the structure.

Nuclear blast pressures are of such great magnitude as to make the design of structures which will resist this type of blast a specialized problem. Considerable information on the design of blast resistant structures is available from the Federal Civil Defense Agency, the Department of Defense, and the U. S. Atomic Energy Commission.

3. Earthquake Forces

Essentially, earthquakes are ground movements resulting from the sudden release of stresses and strains developed in the earth's crust. In most cases, they are the result of the sudden slippage, upheaval, or movement of one section of the earth's crust with respect to another along a fault line. This movement propagates shock waves which pass through the earth, causing movement and vibration of the ground at points far distant from the epicenter.

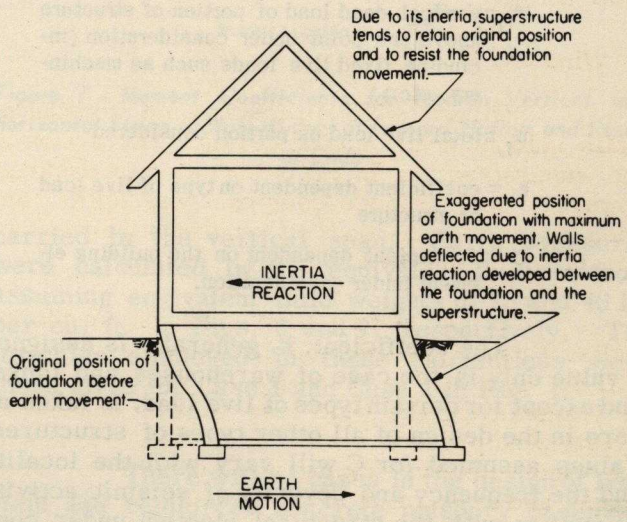

Due to its inertia, superstructure tends to retain original position and to resist the foundation movement.
Exaggerated position of foundation with maximum earth movement. Walls deflected due to inertia reaction developed between the foundation and the superstructure.
INERTIA REACTION
Original position of foundation before earth movement.
EARTH MOTION

Figure 5 - Effect of Earthquake Movements on Buildings and Building Foundations

Ground movements associated with earthquakes consist of both horizontal and vertical ground vibrations. Since the horizontal motion is usually much greater and produces the most destructive force, the vertical movements are usually disregarded. The effect of horizontal ground movements on buildings is illustrated in Fig. 5. As shown in this figure, the building foundation must move with the ground, whereas the superstructure, which is not in contact with the ground, tends to remain at rest in its original position. As the foundation moves, it exerts a force on the superstructure tending to set it in motion, or conversely, the superstructure exerts a force on the foundation tending to keep it in its original position. The force developed is a function of the

acceleration of the horizontal movement and the mass or weight of the superstructure. These two factors have been commonly adopted as measures of the force which buildings may be required to resist if subjected to earthquake movement.

In present-day earthquake design practice, it is customary to assume that the actual reactions produced in a building by an earthquake will be similar to those resulting from a horizontal static force, acting in any direction on the building as a whole or on each principal mass, such as one story of the building. This method is practical, and has been adopted in many building codes. It has proven quite satisfactory even though it is recognized that it is not an exact measure of the actual forces which may be involved. Various formulas have been proposed for determining the assumed horizontal earthquake force for design purposes, however, the one most commonly used and recommended is:

$$F = C(W_D + kW_L)$$

where, F = design lateral force

W_D = vertical dead load of portion of structure above the point under consideration (including fixed live loads such as machinery, etc.)

W_L = total live load on portion considered

k = coefficient dependent on type of live load or structure

C = coefficient dependent on the building element under consideration.

The coefficient, k, generally is assigned a value only in the case of warehouses and tanks, and except for certain types of live load, is taken as zero in the design of all other types of structures. Values assumed for C will vary with the locality and the frequency and severity of seismic activity as well as with the structural element under consideration. In the analysis of the overall stability of a structure, C is usually determined from the formula:

$$C = \frac{0.15A}{N+4.6}$$

In this formula, N is the number of stories in the structure and A is taken as one, two, or four, respectively, for areas subject to infrequent, moderate, or heavy earthquake activity. Higher C values are recommended in the design of parapet walls or similar building appendages, and in the design of tanks, towers, chimneys, and similar structures.

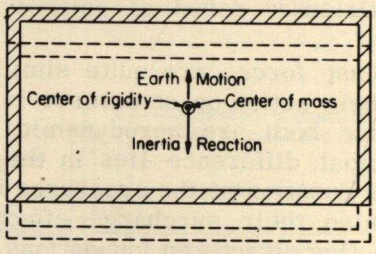
To illustrate the application of these formula, consider the case of a foundation for a one story building located in an area of infrequent seismic activity. The coefficient, k, will be taken as zero in this case and the design lateral earthquake force to be considered as acting at the top of the foundation will be:

$$F = \frac{0.15W_D}{5.6} = 0.027W_D$$

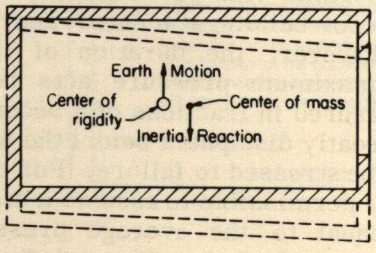
The actual force assumed in the design will be 0.027 times the dead load weight of the entire superstructure including the first floor.

Once the design lateral force has been determined, essentially the same methods as employed in designing for wind forces may be applied in calculating earthquake stresses. As the direction of the earthquake waves cannot be predetermined, the design load is assumed to act in any direction. Since movements which are not parallel to the axes of the building can be resolved into components which do act parallel to the axes, it is usually satisfactory to consider that the force is applied parallel to either of the two axes.

As in the case of wind forces, earthquake forces will tend to cause overturning, translation, and/or rotation of the superstructure with respect to the foundation, and the stresses developed in the foundation walls and in the connections between the foundation and the superstructure should be investigated. It should be noted, however, that whereas wind stresses tend to become more critical as the weight of the construction is decreased in any given design, the reverse is true in the case of earthquake forces. Since the force applied to a structure due to earth movements varies directly with the weight of the structure, stresses due to earthquake forces may become more critical as the weight of the construction is increased. Also, when considering the rotational or torsional effect of earthquake forces. it should



a) Plan View - Basement A



b) Plan View - Basement B

Figure 6 - Rotational Effects of Earthquake Forces on Foundations

be noted that the resultant force will act at the center of mass of the structure, which often will not coincide with the assumed point of application of the resultant of wind forces. The rotational moment acting on the foundation will be equal to the assumed earthquake force times the distance from the center of mass of the superstructure to its center of rigidity, that is, the center of resistance to horizontal translation.

An exaggerated illustration of the effect of earthquake forces in causing rotation is shown in Fig. 6. Where the center of mass and the center of rigidity coincide, as in Fig. 6(a), no rotational moment is introduced. If a portion of one cross wall is removed as in Fig. 6(b), the center of rigidity will be moved more than the center of mass and a rotational moment will be introduced. As a result, the wall furthest from the center of rigidity will be deformed and stressed more highly than the nearer wall.

As previously pointed out in the discussion of wind loads, it is generally assumed that the floor bearing on the foundation will act as a diaphragm in transmitting horizontal shearing forces from the superstructure to the foundation. This also applies to earthquake forces where the action will be similar. The overturning moment will produce compressive and uplift forces on the walls perpendicular to the earth movement. Translation will produce horizontal shearing forces on the parallel walls and rotation will increase the shearing force in some walls, decrease it in others. The effect of translation in causing bending in perpendicular walls should be considered where the first floor will not have sufficient rigidity to provide diaphragm action.

Although the general observations presented herein regarding earthquakes and their effect on structures are applicable to most designs, there are many factors and variables which cannot be covered in detail in this brief discussion. Accordingly, it is suggested that designers interested in earthquake resistant design refer to current published literature on this subject.

C. Moments and Shears

Moments and shears developed in foundation walls due to application of vertical and lateral loads may be calculated by the usual methods. The graphs given in Figs. 8 and 9, pages 10 and 11, present curves useful in estimating moments and shears due to lateral earth pressures acting on foundation walls simply supported at the top and bottom for various wall heights and depths below grade. In plotting these curves, no allowance was made for horizontal distribution of the applied load; that is, it was assumed that all of the load would be

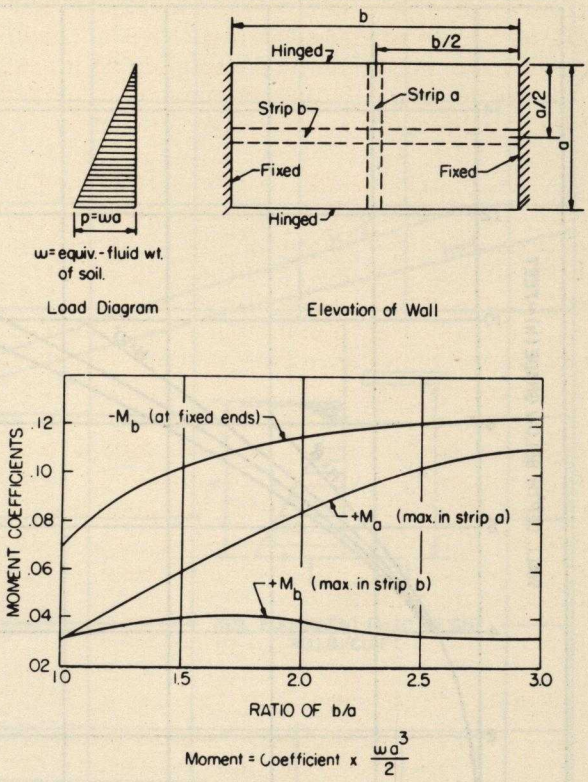


Figure 7 - Moment Coefficients for Median Vertical and Horizontal Strips in Wall Hinged at Top and Bottom and Fixed at Ends

carried by the vertical span. Earth pressures were calculated by the equivalent-fluid method, assuming equivalent-fluid weights of 25 and 40 lb. per cu. ft. in Figs. 8 and 9, respectively. The formulas employed in these calculations were those given for Case 1 and Case 2 in Fig. 1.

There will be cases in the design of both plain and reinforced concrete masonry basement walls where the horizontal moments and shears should be investigated and possibly taken into account. As indicated in Fig. 7, the vertical moment decreases quite rapidly with length to height ratios below about 2.5. The positive horizontal moment is fairly constant, the coefficient varying from 0.032 to 0.042, but its proportion to the vertical moment rises markedly as the length to height ratio decreases. The negative horizontal moment which occurs at the fixed ends is large throughout the entire range of span ratios and in the lower ratios greatly exceeds the vertical moment. Consequently, the assumption that the entire load is carried in the vertical span, while not unsafe, will not necessarily produce the most economical design. In reinforced walls especially, attention should be given to providing sufficient negative reinforcement at fixed ends or over intermediate supports to prevent serious cracking at those points.

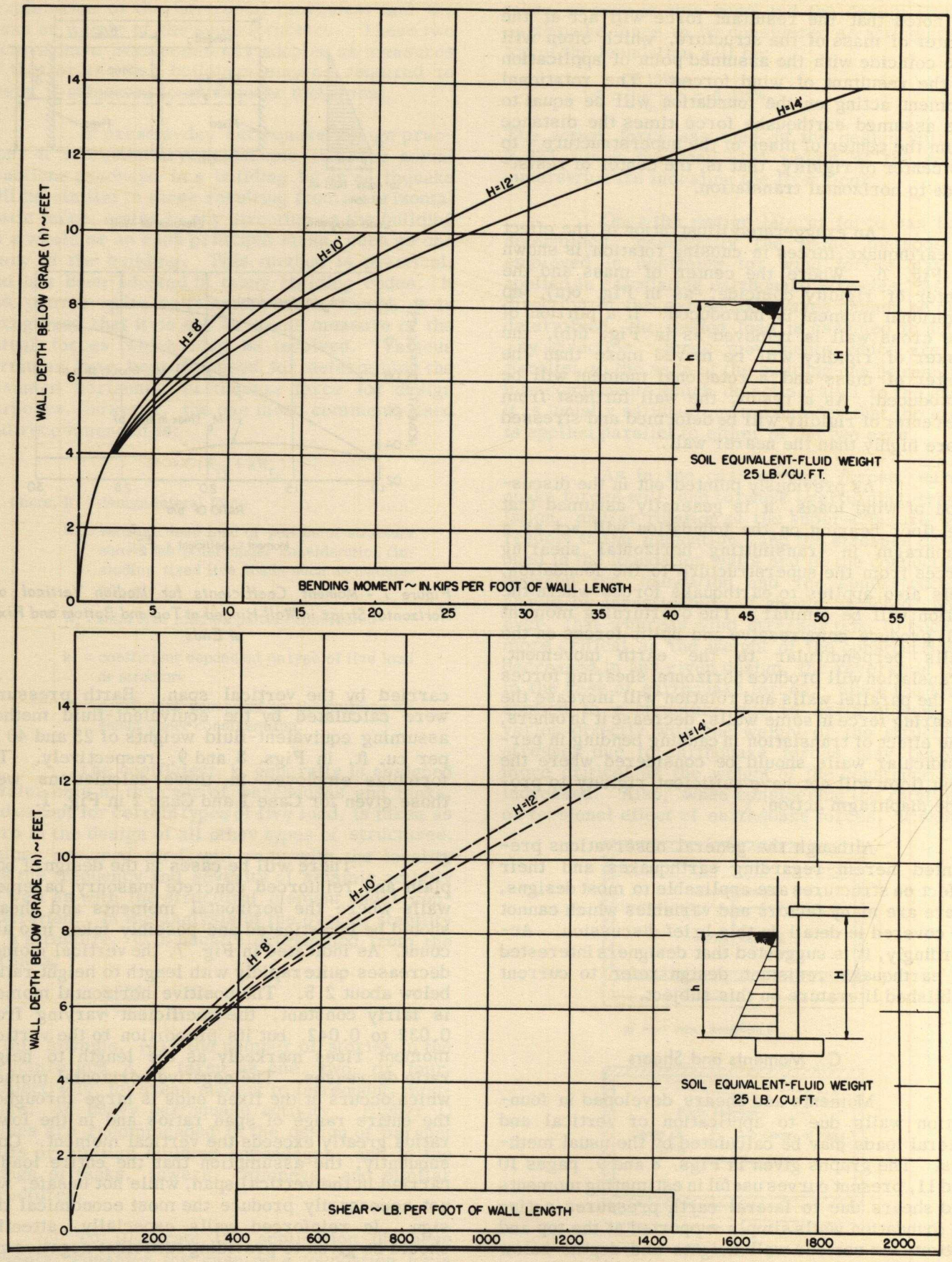


Figure 8 - Bending Moments and Shears in Foundation Walls Resisting Earth Pressure through Bending in the Vertical Span - Type A Soil

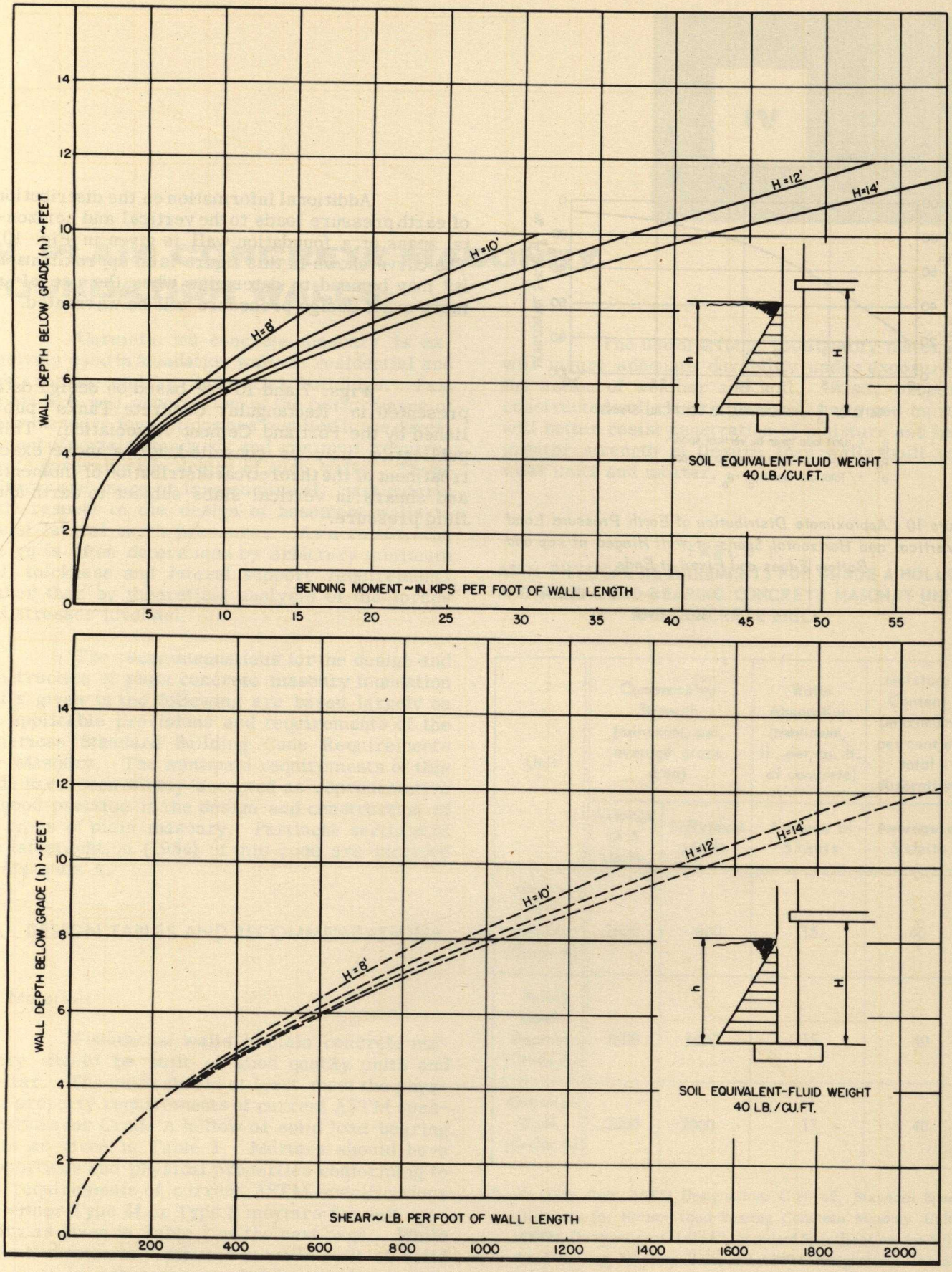


Figure 9 - Bending Moments and Shears in Foundation Walls Resisting Earth Pressure through Bending in the Vertical Span - Type B Soil

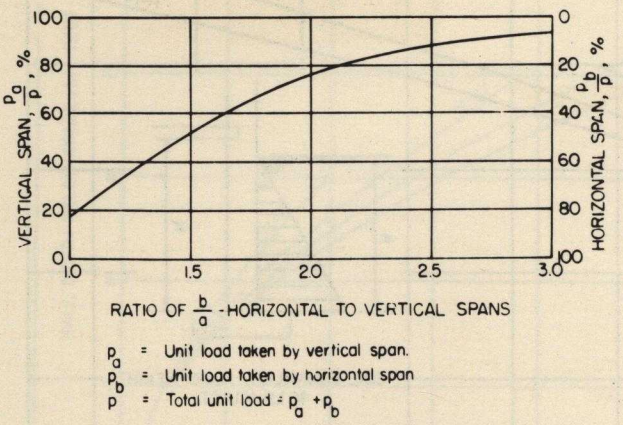


Figure 10 - Approximate Distribution of Earth Pressure Load to Vertical and Horizontal Spans of Wall Hinged at Top and Bottom Edges and Fixed at Ends

Additional information on the distribution of earth pressure loads to the vertical and horizontal spans of a foundation wall is given in Fig. 10. The curve shown in this figure is an approximation but may be used to determine when the use of a more exact design procedure will be warranted.

Figs. 7 and 10 are based on design data presented in "Rectangular Concrete Tanks" published by the Portland Cement Association. This publication may be consulted for a more exact treatment of the theoretical distribution of moments and shears in vertical slabs subject to earth and fluid pressure.

IV. PLAIN CONCRETE MASONRY FOUNDATION WALLS

Unreinforced concrete masonry is extensively used in foundation walls in residential and light commercial or industrial buildings. Past experience and observation of the performance of plain masonry foundations has resulted in the development of certain standards and minimum requirements governing the design of such walls. These criteria are more or less empirical, particularly with respect to the design of basement walls to resist lateral earth pressures. As a result, the design is often determined by arbitrary minimum wall thickness and lateral support requirements rather than by theoretical analysis of the forces and stresses involved.

The recommendations for the design and construction of plain concrete masonry foundation walls given in the following are based largely on the applicable provisions and requirements of the American Standard Building Code Requirements for Masonry. The minimum requirements of this code have been widely accepted as representative of good practice in the design and construction of all types of plain masonry. Pertinent sections of the latest edition (1954) of this code are included in Appendix A.

A. DESIGN TABLES AND RECOMMENDATIONS

1. Materials

Foundation walls of plain concrete masonry should be built of good quality units and mortar. The units should at least meet the physical property requirements of current ASTM specifications for Grade A hollow or solid load-bearing units as given in Table 1. Mortars should have proportions and physical properties conforming to the requirements of current ASTM specifications for either Type M or Type S mortars for unit masonry as given in Table 2 on the next page. While Type N mortar is permitted in walls built of solid units, it is not recommended herein for use in basement walls because of its low structural bond.

The use of strong, good quality materials will insure adequate durability under exposure to the action of weather and soil. Also, properly constructed walls built with strong units and mortar will better resist penetration of moisture and have greater strength in flexure than walls built with weak units and mortar.

TABLE 1.
ASTM PHYSICAL REQUIREMENTS FOR GRADE A HOLLOW AND SOLID LOAD-BEARING CONCRETE MASONRY UNITS AND CONCRETE BRICK*

Unit	Compressive Strength (minimum, psi, average gross area)		Water Absorption (maximum, lb. per cu. ft. of concrete)	Moisture Content (maximum, per cent of total absorption)
	Average of 5 Units	Individual Unit	Average of 5 Units	Average of 5 Units
Hollow Load-Bearing (Grade A)	1000	800	15	40
Solid† Load-Bearing (Grade A)	1800	1600	15	40
Concrete Brick (Grade A)	2500	2000	15	40

* As taken from ASTM Designation: C 90-52, Standard Specifications for Hollow Load-Bearing Concrete Masonry Units; ASTM Designation: C 145-52, Standard Specifications for Solid Load-Bearing Masonry Units and ASTM Designation: C55-55, Standard Specifications for Concrete Building Brick.

† Units with 75 per cent or more net area.

TABLE 2.
MORTAR REQUIREMENTS *

Mortar Type †	Parts by Volume				Minimum Average Compressive Strength of Three 2-in. Cubes at 28 Days, psi.
	Portland Cement	Masonry Cement	Hydrated Lime or Lime Putty	Aggregate (damp, loose condition)	
M (A-1) ‡	1	1 (Type II)	--	Not less than 2-1/4 and not more than 3 times the sum of the volumes of the cements and lime used.	2,500
‡	1	--	1/4		
S (A-2)	1/2	1 (Type II)	--		1,800
‡	1	--	Over 1/4 to 1/2		
N (B) §	--	1 (Type II)	--		750
§	1	--	Over 1/2 to 1-1/4		

* As taken from ASTM Designation: C 270-54T, Tentative Specifications for Mortar for Unit Masonry. The selection of mortars can be made either by specifying ingredients and proportions or by specifying compressive strength and water retentivity. Mortars meeting the proportion requirements given above and composed of ingredients conforming to applicable ASTM Specifications are assumed to meet the compressive strength requirements listed and to have water retention of not less than 70 per cent when mixed to an initial flow of 100 to 115. It is recommended herein that the property specifications (ASTM C 270) shall apply for mortars containing masonry cement.

† Mortar type designations in parentheses are former type designations which have been temporarily retained to avoid confusion during the transition to the use of the newer designations.

‡ These mortars recommended where high structural bond is required as in basement walls.

§ Type N mortar not recommended for use in hollow unit foundations or in basement walls built of either solid or hollow concrete masonry units.

In Table 2, certain mortars are recommended over others because of their demonstrated superior structural bond with concrete masonry units, a very important factor in the flexural resistance of basement walls to backfill pressures. A mortar consisting by volume of one part portland cement, one-half part hydrated lime and four to four and one-half parts sand has proved to be especially well suited for use in concrete masonry foundations.

Where the lime hydrate is not of the high calcium type, it is recommended that it be Type S, ASTM C207. The limitation on the unhydrated oxide content of Type S lime is considered a desirable precaution against possible delayed expansion of mortars containing lime, especially the dolomitic type.

2. Allowable Stresses

Recommended working stresses in compression for unreinforced concrete masonry walls are given in Table 3. These working stresses apply to walls laid up to meet average standards of workmanship and with face shell mortar bedding, and have been so selected as to provide a minimum safety factor of about 3.5 with axial loading. They are more conservative with respect to walls laid up with full mortar bedding and superior workmanship.

It should be noted that the recommended working stresses in Table 3 refer to wall stresses resulting from either uniformly distributed or concentrated loads. When designing for concentrated loads, it usually will be permissible to employ a higher working stress in bearing directly under the load reaction -- an allowable bearing stress 50 per cent greater than the working stress in vertical compression being commonly used and even higher stresses under some conditions. It should also be noted that the actual wall stresses due to concentrated loads will be dependent upon the manner in which the load is distributed over the wall length and thickness. Generally, closely spaced concentrated loads, such as those from typical joist-type floor systems, may be converted to an equivalent uniform load and wall stresses calculated accordingly. Isolated or widely spaced concentrated loads present a special problem, since it is obvious that wall stresses due to such

TABLE 3.
ALLOWABLE COMPRESSIVE STRESSES AND AXIAL COMPRESSIVE LOADS ON UNREINFORCED CONCRETE MASONRY FOUNDATION AND BASEMENT WALLS *

Type & Grade of Unit	Mortar Type	Allowable Compressive Stress, psi. (gross section) †	Allowable Axial Compressive Load, lb. per lin. ft. †			
			8" Wall	10" Wall	12" Wall	16" Wall
Solid - Grade A	M (A-1)	175	16,000	20, 200	24, 400	32, 800
	S (A-2)	160	14, 600	18, 400	22, 300	30, 000
	N (B)	140	12, 800	16, 100	19, 500	26, 200
Hollow-Grade A	M (A-1)	85 ‡	7,750	9, 800	11, 800	15, 900
	S (A-2)	75 ‡	6,850	8,650	10, 400	14,050

* Allowable stress values are as recommended in the American Standard Building Code Requirements for Masonry (National Bureau of Standards Miscellaneous Publication 211, issued July 15, 1954.)

† Allowable stress and load values for hollow unit walls given in this table are for construction in which the ratio of the mortar bedded area to the gross area will range between 0. 25 to 0. 33 based on the minimum face shell and web thicknesses. Where the wall design is such that the ratio of the mortar bedded to the gross area will be below or above these values, the design allowable stresses and loads should be adjusted accordingly.

‡ Allowable compressive stresses of 105 psi and 95 psi on the gross section are recommended in the design of piers and pilasters built of hollow units with the cellular spaces filled solidly with grout or concrete and with Type M and Type S mortar, respectively.

loads will be highest directly under the load and will diminish with distance outward and downward from the load. Where walls are to be laid up in common or running bond, it is a common practice to assume that isolated concentrated loads will be distributed outward and downward through a pyramid of masonry, as shown in Fig. 11, with sides sloping not less than 30 degrees nor more than 45 degrees from the vertical. The wall stress at any level is then calculated by dividing the total load by the base area of this pyramid. Another practice, specified in some building codes, is to base the load distribution assumed in calculating wall stresses on either the width of the wall or the length of the individual wall units. The Uniform Building Code, for example, specifies that concentrated loads be considered as distributed over a wall length not exceeding the width of the bearing plus four times the wall thickness with solid masonry built of solid units; or over a wall length not greater than one and one-half times the length of the wall units with walls built of hollow units. In the design of cavity walls or walls with continuous vertical joints, stacked bond construction for example, this same code requires that concentrated loads be assumed distributed over a wall length not greater than the length of the wall units.

Whereas the allowable stresses in compression given in Table 3 have been widely accepted by most authorities and building officials, the same is not true with respect to working stresses in tension and shear in plain masonry. It is generally recognized that the flexural strength of plain masonry in bending in the vertical span will be dependent principally upon the tensile and shear bond developed between the masonry units and the mortar. This also is true of the horizontal flexural strength of walls laid in stacked or similar bond

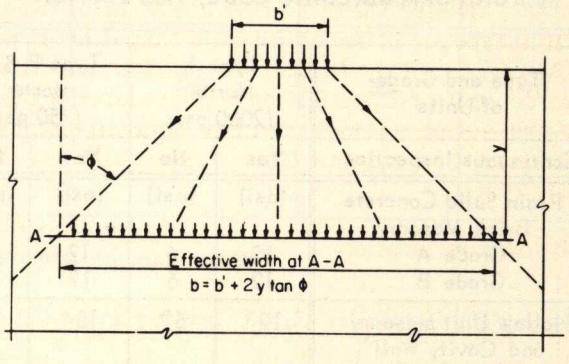


Figure 11 - Assumed Distribution of Concentrated Loads in Walls Where Units are Laid in Common or Running Bond.

patterns and, as a result, containing continuous vertical joints. On the other hand, the horizontal flexural strength of walls laid in running bond will depend more upon the design and strength of the concrete units than upon the mortar bond developed in either the horizontal or vertical joints. Generally, walls laid in running bond will have greater flexural strength in the horizontal span than will stacked bond walls or either running or stacked bond walls in the vertical span. For this reason, stacked bond construction is seldom recommended or used in walls which must be designed to resist lateral forces unless required for architectural reasons, in which case it may be used if reinforced horizontally with either bond beams or joint reinforcement.

Due to the many variables affecting mortar bond and the relatively low strength of masonry in tension, most authorities have considered it prudent to neglect the tensile resistance of plain masonry when designing foundation walls. Where this practice is followed, the design is based on the assumption that the principal bending will be in the vertical span and that vertical dead load compressive stresses will be present and act to prevent development of or greatly reduce any vertical tensile stress. In addition, arbitrary minimum wall thickness and lateral support requirements based on empirical data obtained through observation and experience help to insure provision of an adequate wall section.

Although, for the ordinary building load and design conditions, plain masonry walls may be safely and satisfactorily designed through use of arbitrary minimum requirements, such as those given in the American Standard and other building codes, special load or other design considerations may be encountered which require calculation of the flexural strength of the walls by theoretical methods. In such cases, it will be necessary for the designer to have some knowledge of the ultimate strength properties of the masonry in bending or to make some assumption as to safe working stresses. Accordingly, working stresses in flexural tension and shear for plain concrete masonry, as recommended in the Uniform Building Code, are shown in Table 4. Data relative to the ultimate flexural and shearing strength of plain concrete masonry walls and the ultimate tensile and shearing bond strength developed between various mortars and concrete masonry units are given in Tables 5 to 9. While these data are not extensive, they will be of assistance to designers in the selection of safe working stresses for design purposes. These data, considered with respect to the mortars recommended herein, provide substantial evidence that the ultimate flexural strength of plain concrete

TABLE 4.
ALLOWABLE STRESSES IN FLEXURAL TENSION AND SHEAR IN UNREINFORCED CONCRETE MASONRY - UNIFORM BUILDING CODE, 1955 EDITION

Type and Grade of Units	Type A Mortar (2000 psi)		Type B & C Mortar (750 psi)	
Continuous Inspection	Yes	No	Yes	No
Plain Solid Concrete Brick Masonry	(psi)	(psi)	(psi)	(psi)
Grade A	12	6	12	6
Grade B	12	6	12	6
Hollow Unit Masonry and Cavity Wall Masonry	12*	6*	10*	5*

* On net area

TABLE 5.
FLEXURAL STRENGTH OF CONCRETE MASONRY WALLS IN THE VERTICAL SPAN *

Wall Construction †				Modulus of Rupture, psi ‡	
No. of Wall Panels Tested	Units: Aggregate Type	Units: Compressive Strength, psi. (gross area)	Mortar Compressive Strength, psi §	Range	Avg.
11	Sand & Gravel Cinders Haydite Slag	820 - 1290	445 - 940	13-42	24
9	Sand & Gravel Cinders Lime-stone	600 - 1550	1010 - 1440	18-47	29
4	Sand & Gravel Cinders	550 - 1570	1860 - 2200	18-36	25

* Data from University of Illinois Bulletin No. 251 and National Bureau of Standards BMS Reports 5 and 32.

† All walls were built of hollow units laid in running bond pattern and tested at an age of one month. Majority of walls were laid with full mortar bedding, however, results reported for six walls laid with face shell bedding are included. Walls were simply supported at top and bottom edges and a concentrated load applied either at midpoint or at outer quarter points of of span.

‡ Solid rectangular section assumed in calculating values of modulus of rupture given in the table. Values based on net section would be 25 to 40 percent higher.

§ Mortar mixtures, by dry rodded volumes, were 1:1:4-1/2 and 1:0.25:3 mixtures of portland cement, hydrated lime and sand.

TABLE 6.
FLEXURAL STRENGTH OF PLAIN CONCRETE MASONRY WALLS IN THE HORIZONTAL SPAN *

Wall Construction †					Ultimate Resisting Moment, in. lb./ft.	Modulus of Rupture, psi	
No. of Wall Panels Tested	Wall Thickness, in.	Units		Mortar Compressive Strength, psi		Based on Assumed Solid Section	Based on Actual Hollow Section
		Condition When Laid	Compressive Strength, psi (gross area)				
3	4	Air-dry	1370	500	8370	320	350
9	8	Air-dry	1720	2740	18900	160	225
				500	18900	160	225
				290	17100	145	205
9	8	Damp	1720	2740	17700	150	210
				500	17700	150	210
				290	19860	170	235
3	12	Air-dry	1330	500	21000	80	135

* Data from tests conducted at Iowa State College for the National Concrete Masonry Association.

† All walls (4 ft. high by 6 ft. 8 in. long) were built with three-core hollow expanded shale aggregate units laid in running bond with face shell mortar bedding. Tests were conducted at 28 days with walls in the vertical position, simply supported at the sides, free at the top and bottom edges, and with an applied compressive load of 10 psi on the gross area in the vertical direction. Test load applied at the midpoint of the horizontal span.

SPECIAL NOTE: Modulus of rupture and resisting moment values shown in the above table will not be similar for all block walls. The horizontal flexural strength of plain walls will vary depending upon the tensile strength, compressive strength, and design of the units and other factors.

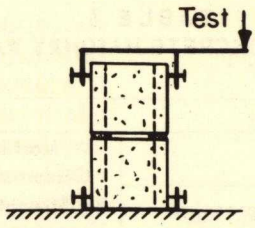
TABLE 7.
DATA ON THE RACKING OR HORIZONTAL SHEARING RESISTANCE OF CONCRETE MASONRY WALLS *

Wall Construction †					Ultimate Horizontal Shearing Stress, psi	
Wall Thickness, in.	Units			Mortar Compressive Strength, ‡ psi	Gross Area	Net Area
	Size, in.	Aggregate Type	Compressive Strength, psi (gross area)			
8	8 x 8 x 12	Sand & Gravel	1190	445	35	70
6	6 x 8 x 16	Expanded Slag	1020	600	25	67

* Data from tests conducted at the National Bureau of Standards and reported in BMS Reports 5 and 32.

† Eight in. wall laid up with two-core units laid in running bond and with full mortar bedding. Six in. wall laid up with three-core hollow units laid in running bond and with face shell mortar bedding.

‡ 1:1:6 portland cement - hydrated lime - sand by volume using damp, loose sand.



Tensile Bond Test Method

TABLE 8
TENSILE BOND STRENGTH DEVELOPED BETWEEN MORTAR AND CONCRETE MASONRY UNITS AS DETERMINED IN ECCENTRIC LOAD TESTS OF TWO-BLOCK PIERS *

No. of Piers Tested †	Mortar ‡							Ultimate Flexural § Tensile Bond Strength, psi	
	Type (ASTM-C270)	Composition				Compressive Strength, psi	Tensile Strength, psi	Range	Average
		PC	MC	L	S				
Mortars Recommended for Concrete Masonry Foundations									
67	M (A-1)	1	-	1/4	3	2540-3150	330-350	43-105	66
15	M (A-1)**	1	1	-	6	1330-1600	205-250	20- 36	28
15	S (A-2)	1	-	1/2	4-1/2	1340-1560	180-200	87-108	98
Weighted Average									65
Mortars Not Recommended for Use in Foundations									
102	N (B)	1	-	1	6	500-1040	110-165	12- 56	27
37	N (B)	-	1	-	3	330- 600	60-134	4- 13	7
37	O (C)	1	-	2	9	290- 400	45- 85	6- 18	9

* Data from tests conducted at the Univ. of Wisconsin, Iowa State College and the Univ. of Toledo for the National Concrete Masonry Association.

† All piers built with hollow sand and gravel aggregate units and face shell mortar bedding and tested at 28 days age.

‡ PC-portland cement; MC-masonry cement (Type II); L-hydrated lime; S-mortar sand (FM 1.85-1.97). Mortars were mixed to initial flows ranging between 120 to 140.

§ Average stress in mortar joint in tension at failure based on the net area bedded in mortar. The range reported is the range in the average values obtained with 5 or 6 pier specimen per variable.

** This mortar mix recommended only where it complies with the property specification for Type M (A-1), ASTM C 270.

masonry walls will be well in excess of the calculated strength based on the allowable working stresses in Table 4.

It will be noted that in the tests summarized in Table 6, mortar strength was not an important factor in determining the horizontal flexural resistance of the walls; from which it may be deduced that the tensile strength of the face shells and the shearing strength of the cross webs are the important factors. In contrast, the data presented in Table 8 indicates that tensile bond, which virtually establishes the vertical flexural resistance, is greatly influenced by the composition and strength of the mortar and, probably, other factors as well. Among these other properties, data from these and other investigations leave little doubt that the consistency of the mortar has a very

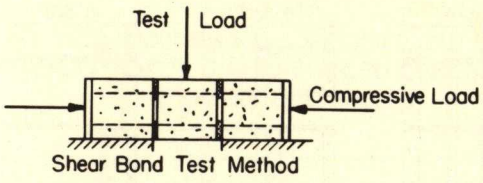


TABLE 9.
SHEAR BOND STRENGTH DEVELOPED BETWEEN MORTAR AND CONCRETE MASONRY UNITS AS DETERMINED IN TRANSVERSE LOAD TESTS OF THREE-BLOCK PIERS *

No. of Piers Tested †	Mortar				Compressive Load on Piers, psi, (gross area)	Ultimate Shear Bond Strength, psi, (net mortar bedded area)	
	Composition ‡ (proportions by loose volume)			Compressive Strength, psi			
	PC	L	S			Range	Average
12	1	1/4	3	3150 & 2630	0	55- 72	61
4				2630	5	84- 93	90
3				2630	10	89-106	99
7	1	1	6	820 & 800	0	33- 38	36
4				800	5	68- 77	72
3				800	10	76- 85	80
3	1	2	9	400	0	23- 27	24
4				400	5	50- 60	55
3				400	10	59- 66	63

* Data from tests conducted at the Univ. of Wisconsin and the Univ. of Toledo for the National Concrete Masonry Association.

† Piers were built of hollow sand and gravel aggregate units laid with face shell mortar bedding and tested at 28 days age.

‡ PC-portland cement (Type I); L-hydrated lime (Type S-dolomitic); S-mortar sand (FM 1.85-1.97). Mortars were mixed to an initial flow of 140 in the Univ. of Wisconsin tests and 130 in the Univ. of Toledo tests.

large influence on its capacity to adhere strongly to concrete units. Relatively stiff and "dry" mixtures bond poorly, especially when used with units of low moisture content or high suction properties. In general, it has been found that high structural bond between the mortar and units requires not only that the mortar be of requisite strength but that its consistency be highly plastic and relatively "wet," corresponding to a flow in the range of 120 to 140.

Considering the mortar mixtures recommended herein for use in foundation construction, mixed to a flow of 120 to 140, it is believed that the following ultimate structural bond values in tension and shear can be safely assumed:

Modulus of rupture = 60 psi on net section
Shear (ultimate) = 85 psi on net section

The use of higher working stresses in flexural tension and shear than recommended in Table 4 may be justified in the case of walls built with high strength, high bond mortars and average or better workmanship. Also, most codes permit working stresses to be increased by one-third when designing for wind, earthquake, or blast forces, provided the stresses due to dead and other live loads alone do not exceed the basic allowable values. Where higher working stresses are assumed, precautions should be taken to insure that the assumed quality of the materials and workmanship will actually be employed in the construction.

3. Recommended Minimum Wall Thickness

The nominal thickness of plain concrete masonry foundation walls should be not less than 8 in. nor less than that required to support the vertical compressive and lateral loads without exceeding the allowable stresses. Normally, concrete masonry walls are most practical and economical when designed and built to nominal wall thicknesses of 8, 10, or 12 in., since practically all block manufacturers produce and stock standard units in these widths. Where 16 in. wall thicknesses are required, they may be obtained either by laying standard length units in the wall endwise, or by using more than one unit to make up the wall thickness, as for example, two standard 8 in. units or 12 and 4 in. units. Walls thicker than 16 in. may also be obtained but seldom will be economical. In such cases, consideration should be given to the use of reinforced concrete masonry in lieu of plain masonry construction.

Typical 8, 10, 12 and 16 in. thick foundation walls built of hollow concrete masonry units are shown in Fig. 12 on the following page. Recommended allowable compressive loads for these wall thicknesses are given in Table 3, page 15, and allowable wall heights and depths below grade in Table 10, page 21, for both solid and hollow unit construction.

The wall heights and depths below grade recommended in Table 10 are for average conditions and presume earth pressures and compressive dead loads as normally encountered in light building construction. Since no allowance is made for the effect of variations in the compressive dead load carried by the walls on their bending resistance in the vertical span, these recommended wall heights and depths will become increasingly conservative as the dead load is increased and correspondingly less conservative as it is decreased. The graph in Fig. 13, page 20, illustrates the theoretical increase in the vertical bending resistance of 8, 10 and 12 in. hollow unit walls accompanying an increase in axial compressive load of from 0 to 2000 lb. per lin. ft. It can easily be seen from this graph that

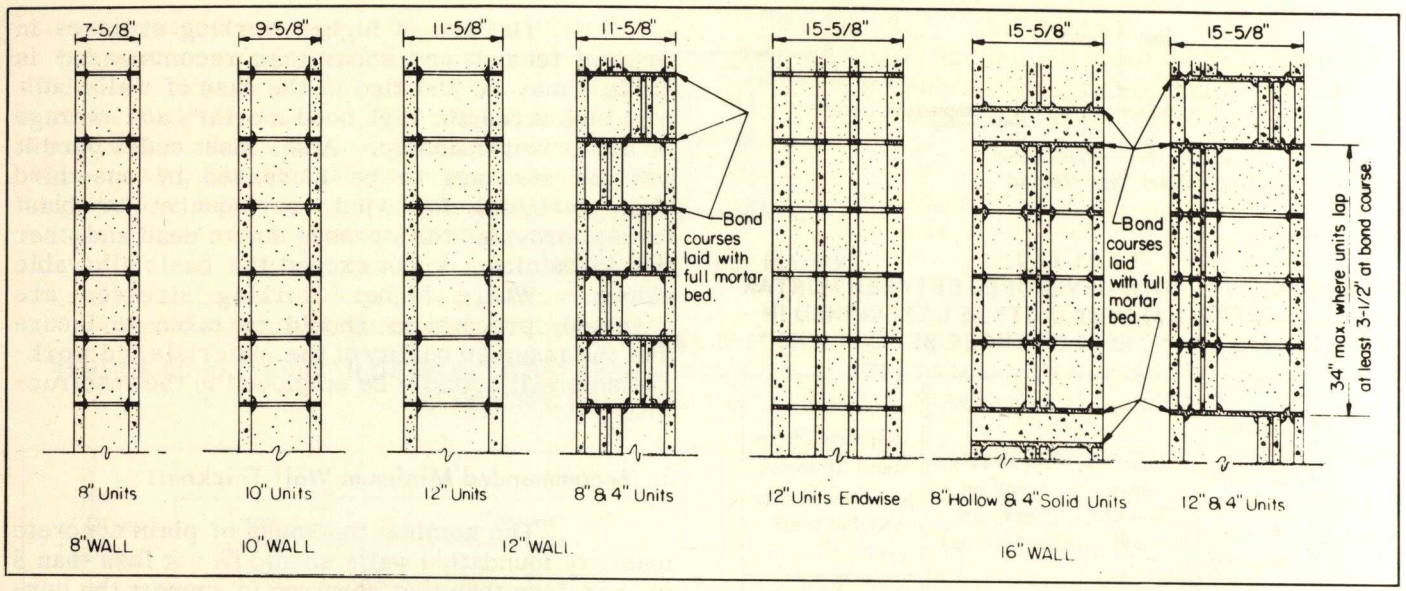


Figure 12 - Typical Hollow Unit Concrete Masonry Wall Sections.

the use of thinner foundation walls for light structures than for heavy structures, as permitted in some codes, is not justified from the standpoint of lateral stability against earth pressure. Also, where basement walls are backfilled prior to erection of the superstructure, it is evident that bending stresses will be most critical at the time of backfilling since only the wall dead load will be present to reduce the flexural tensile stresses due to backfill pressure.

Walls built to heights, thicknesses and depths below grade as recommended in Table 10 often are backfilled prior to erection of the superstructure without the occurence of wall failures. This fact cannot be theoretically explained if it is assumed that plain masonry has no value in resisting tensile stress. Furthermore, comparison of the calculated resisting moments given in Fig. 13 for walls carrying compressive dead loads of between 200 to 300 lb. per lin. ft., with the bending moment values given in Fig. 7 for wall heights and depths below grade as recommended in Table 10, would indicate that such walls would not have sufficient strength to safely resist the lateral earth pressures. The fact that walls of such design have been built and have, for many years, performed satisfactorily without failing under lateral loads greater than here assumed again would tend to indicate that plain concrete masonry is capable of withstanding some tensile stress and that a significant portion of the load is carried by the horizontal span. In any case, it is evident that care should be taken to insure that foundation walls of plain masonry are built in such a way as to provide the greatest possible tensile resistance even when it is theoretically neglected. Available data indicates that the structural bond developed between mortar and concrete masonry units is largely dependent upon the strength and consistency of the mortar, other variables remaining constant, and accordingly, it is again emphasized that a strong, plastic mortar should be used in the construction of concrete masonry foundation walls.

4. Lateral Support

The recommended minimum wall thicknesses and allowable depths below grade for plain masonry foundation walls given in Table 10 are based on the assumption that lateral loads will be

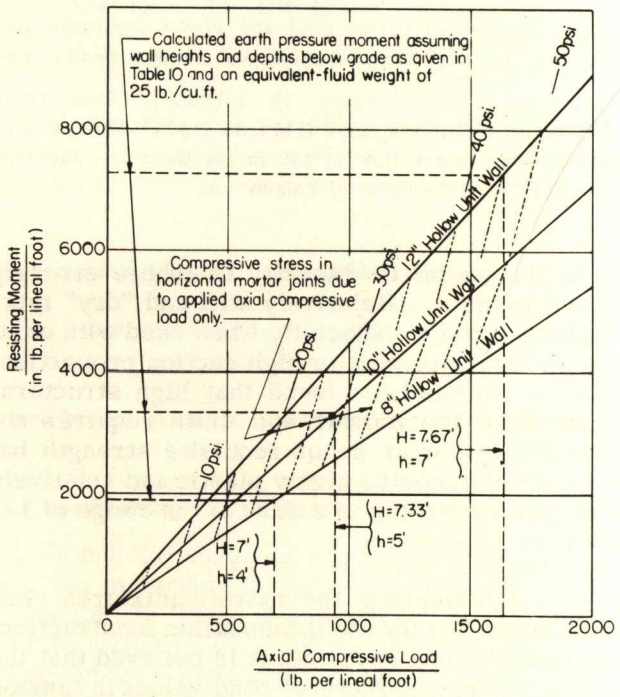


Figure 13 - Effect of Axial Compressive Loads in Increasing the Bending Resistance of Hollow Unit Concrete Masonry Walls in the Vertical Span.

TABLE 10.
RECOMMENDED MAXIMUM WALL HEIGHTS AND DEPTHS BELOW GRADE FOR UNREINFORCED CONCRETE MASONRY BASEMENT WALLS

Wall Construction	Wall Thickness	Recommended Maximum Values *	
		Wall Height (H)	Depth Below Finished Grade (h)
Solid Units	8"	7'-0"	5'-0" †
	10"	7'-8"	6'-0" †
	12"	8'-4"	7'-0"
	16"	9'-0"	8'-4"
Hollow Units	8"	7'-0"	4'-0" †
	10"	7'-4"	5'-0" †
	12"	7'-8"	7'-0"
	16"	8'-0"	7'-8"

* Where basement walls will be laterally supported by the first floor at the top and the footing or basement floor at the bottom, the clear vertical span (H) between such lateral supports and the depth below finished grade (h) should not exceed the values given except as noted below. With clear vertical spans and/or depths below grade exceeding those given in the table, a special design will usually be required.

† Allowable depths below grade for 8 in. and 10 in. walls may be increased to a maximum of 7 ft. 0 in. where favorable soil conditions permit and with the approval of the local building official or other competent and authorized parties. Where such increased depths are permitted with walls exceeding 30 to 35 ft. in length between vertical lateral supports, consideration should be given to the need for additional lateral support.

carried by the walls through bending in both the horizontal and vertical spans. In the horizontal span, lateral support is furnished by crosswalls at building corners, intersecting cross-partitions, and pilasters, columns or similar vertical members. Lateral support at the top of basement walls generally is furnished by the first floor construction and at the bottom by the wall footing or basement floor slab.

The effect of crosswalls, partitions and offsets in reducing the horizontal spans and thereby increasing the bending resistance of basement walls is illustrated in Fig. 14. As shown here, the maximum horizontal wall span between supports in Basement A, at the top of the figure, is 38 ft.; whereas in Basement B, in the lower half, it is 15 ft. It is evident that Basement B will be more rigid and will withstand considerable more lateral load than Basement A even though the overall wall lengths are the same in both cases.

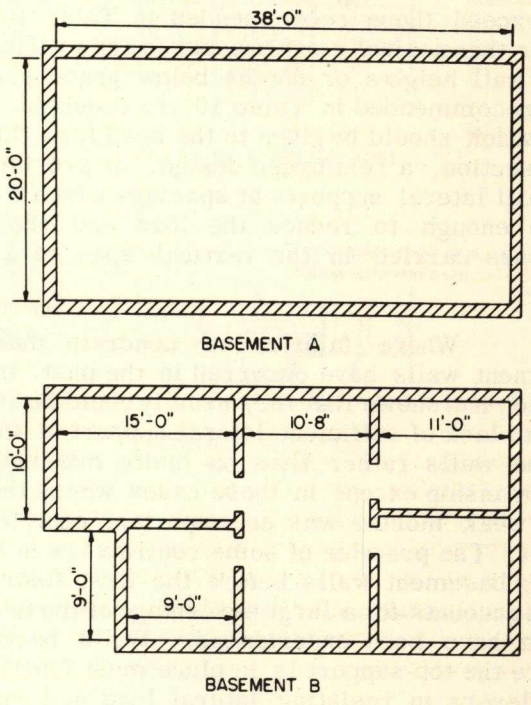


Figure 14 - Effect of Crosswalls and Offsets in Reducing Wall Spans in Horizontal Bending.

When considering the effect of vertical and horizontal lateral supports on the stresses developed in a hypothetical basement wall of homogeneous, isotropic construction, it is important to remember that the distribution of the earth pressure load to the vertical and horizontal spans will vary with the span ratio approximately as shown in Fig. 10, page 12. Assuming a wall panel of given height and supported at the top and bottom, it is evident that the load carried in the horizontal span will decrease quite rapidly as the spacing between vertical lateral supports is increased. The exact distribution of the load will, of course, be affected by other factors such as the shape or pattern of the load diagram, the degree of fixity developed at each edge support, and the relative stiffness of the wall in the two directions. For example, common bond masonry generally will be stiffer in the horizontal than in the vertical span, and assuming equal span lengths and support conditions in both directions, that portion of the load carried in the horizontal direction will tend to be greater than that carried in the vertical direction. However, for practical design purposes, the horizontal distribution becomes negligible and may be neglected

when the ratio of horizontal to vertical spans exceeds about two or three.

Past experience has indicated that plain concrete masonry walls, supported at the top and bottom, will have sufficient flexural strength to safely carry all of the lateral earth pressure load which may be expected with average soil conditions where the wall heights and depths below grade do not exceed those recommended in Table 10. If higher than normal earth pressures are anticipated, or if wall heights or depths below grade greater than recommended in Table 10 are required, consideration should be given to the need for a thicker wall section, a reinforced design, or provision of vertical lateral supports at spacings which will be close enough to reduce the load and the wall stresses carried in the vertical span to a safe level.

Where failures of concrete masonry basement walls have occurred in the past, investigation has shown that they usually could be attributed to lack of sufficient lateral support at the top of the walls rather than to faulty materials or workmanship except in those cases where the use of a weak mortar was an important contributing factor. The practice of some contractors in backfilling basement walls before the first floor is in place accounts for a large percentage of the failures which have been experienced. Walls backfilled before the top support is in place must function as cantilevers in resisting lateral load and sustain appreciably higher flexural stresses than would be expected when supported at the top. The difference in the shear and moment developed in walls acting as simple beams as compared to cantilever walls, all other conditions being equal, is illustrated in Fig. 15. In this particular case, it may be seen that the maximum moment and shear values for the cantilever wall will be 2.3 and 2.4 times the values for a wall simply supported at the top and bottom.

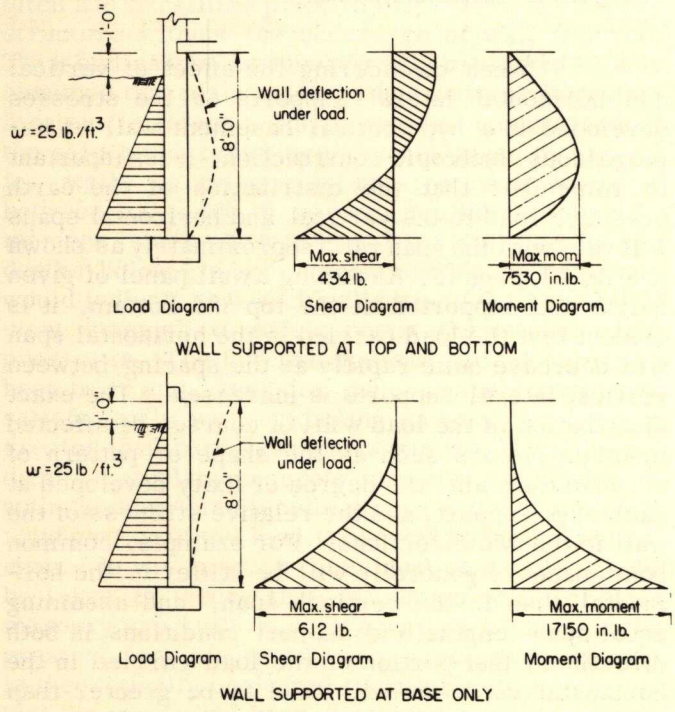


Figure 15 - Effect of Backfilling before Top Support is in Place on Moment and Shear in Foundation Wall.

It appears evident that backfilling against basement walls before they are restrained at the top cannot always be avoided and often may be desirable from the contractors standpoint. Such backfilling practices should never be recommended however, and in those cases where early backfilling cannot be avoided, consideration should be given to provision of temporary vertical and horizontal bracing until such time as the permanent support for the top of the walls is in place. Under certain conditions, temporary bracing may also be desirable during and after backfilling of walls supported at the top but not carrying the full superstructure dead load, particularly where bulldozers or other heavy equipment are used to place the backfill, where the backfill is wet or saturated when placed, or when heavy rains are expected shortly after the backfill is placed and before it has had a chance to consolidate. In the latter case, rain water percolating through and saturating the unconsolidated backfill can result in pressures on the wall far exceeding that with the earth in a dry or normal state.

5. Pilasters

Pilasters, essentially thickened wall sections, are often used to support heavy concentrated vertical loads and to provide lateral support for basement walls. They may be built of hollow or solid units, filled cell hollow unit, or reinforced filled cell hollow unit masonry. A few typical pilaster designs are shown in Fig. 16.

The design of pilasters of hollow or solid units or of filled cell hollow unit construction differs very little from the design of similar wall constructions. In compression, the same working stresses as employed in the design of walls are used except that somewhat higher values, 105 psi and 95 psi on the gross area, are permitted for pilasters and piers of filled cell construction laid up with Type M and Type S mortar respectively. The lateral load resistance of unreinforced pilasters can be calculated by theoretical methods but, in practice, arbitrary rules or code requirements are generally followed in selecting the dimensions and spacing of pilasters which serve primarily to stiffen the wall against lateral load. A suggested "rule of thumb" is that the pilaster projection should be not less than one-twelfth the pilaster

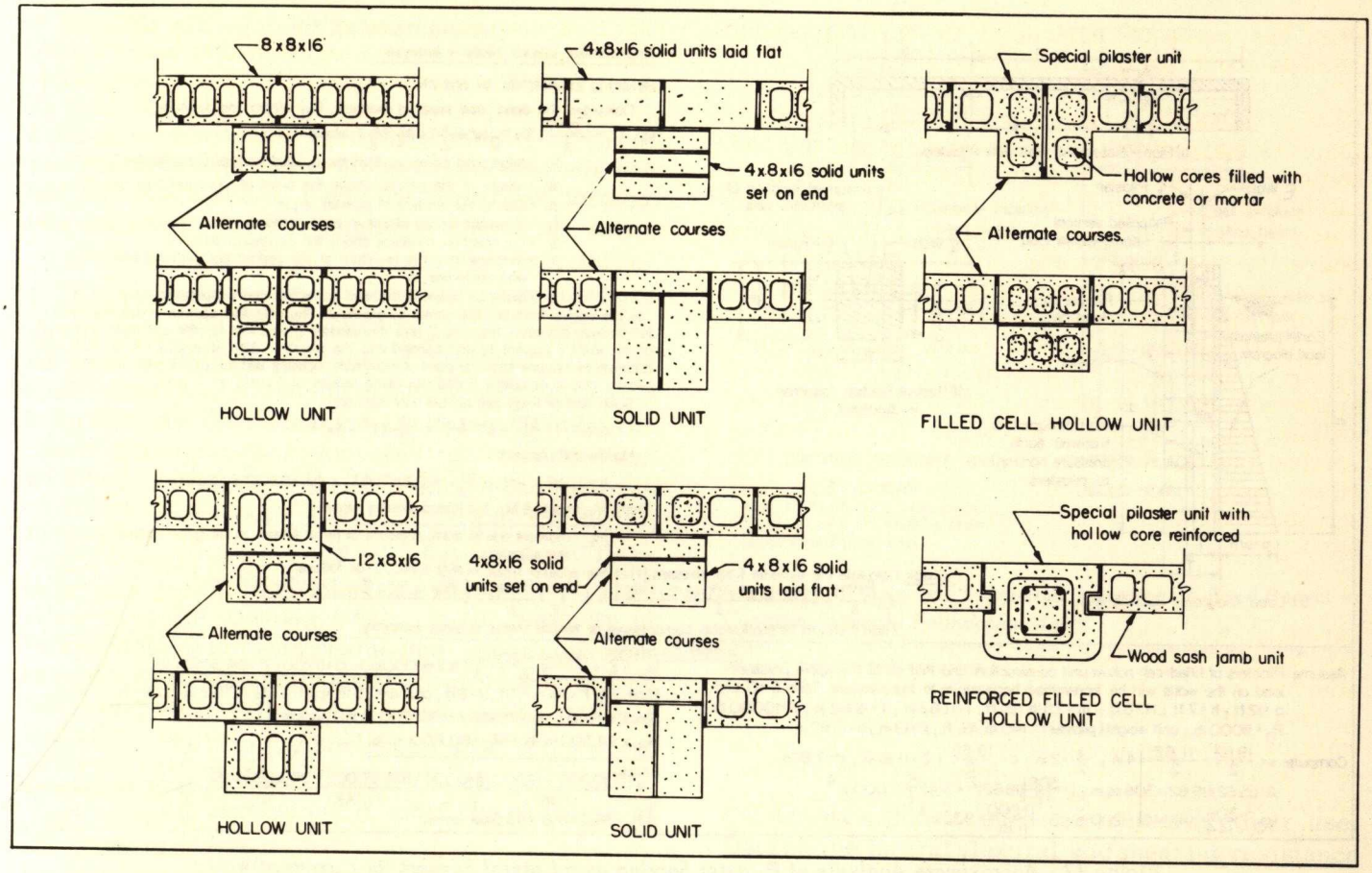
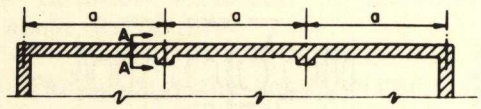


Figure 16 - Typical Concrete Masonry Pilaster Designs.

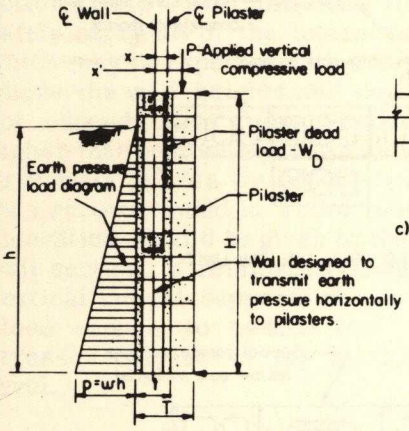
height between supports and the width not less than one-tenth the wall length between pilasters or other vertical supports. For example, in a wall 8 ft. in height between supports, the pilaster projection should be not less than 8 in. Similarly, if the pilasters are to be spaced 13 ft. 4 in. o.c., the pilaster width should be not less than 16 in.

When applying theoretical methods to the design of plain masonry pilasters, it is important to remember that the stiffening effect and the increase in bending resistance afforded by pilasters will be a function of both the vertical compressive load carried by the pilaster and the increase in the moment of inertia of the wall and pilaster combination. Where pilasters are used in walls carrying relatively light compressive loads and not laterally supported at the top, a major portion of the lateral load carried by the walls will be transmitted horizontally to the pilasters or other lateral supports which, in turn, must transmit this load vertically to the footing and the floor or roof diaphragm. Pilasters with dimensions as ordinarily employed will not have sufficient bending resistance to serve as full lateral supports in such cases unless carrying substantial vertical dead load, and such structural action by the pilasters should not be assumed unless the vertical dead load will be sufficient to reduce to a safe level or prevent development of any flexural tensile stresses on the pilaster section. An approximate method of analysis for determining the lateral stability of pilasters serving as lateral supports for basement walls is given in Fig. 17 on the next page. Pilasters not supporting a substantial compressive load generally should be presumed to function as stiffeners rather than as full supports, and be considered as simply adding to the stiffness and bending resistance of the walls in the vertical span. An approximate method of analysis for such cases is given in Fig. 18.

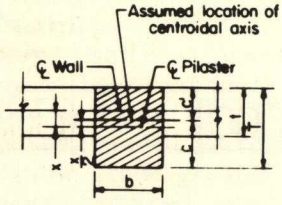
Reinforced filled cell concrete masonry pilasters are essentially reinforced concrete members and may be designed by the methods and procedures normally used in the design of reinforced concrete but with working stresses as recommended in the following section of this booklet. Pilasters of such construction are readily designed to serve as full lateral supports.



a) Plan – Basement Wall with Pilasters



b) Load Diagram - Section A-A



c) Effective Section Assumed in Analysis

Approximate Lateral Stability Analysis

Referring to sketches b) and c):

Calculate the dead load resisting moment, M_R, from the formula,

$M_R = (f_t + \frac{P_D + W_D}{A})S + P_D(x' - \frac{x}{2}) + W_D(\frac{x}{2})$, where

P_D = dead load component of the applied vertical compressive load, lb.
W_D = weight of the pilaster above the point of maximum moment, lb.
A = area of net section of pilaster, sq. in.
f_t = allowable tensile stress in plain masonry, psi.
$S = I/c$ = section modulus about the centroidal axis, $in.^3$
x' = distance from the resultant of the applied compressive load to the wall centerline, in.
x = distance between the wall centerline and the pilaster centerline, in.

In the above formula, the center of gravity of the effective section is assumed to fall midway between the wall ℄ and the pilaster ℄ to allow for the stiffening effect of the walls adjacent to and bonded into the pilaster. Also, it usually will be sufficiently accurate to assume that the point of maximum moment will fall at the mid-height of the pilaster and to calculate S and M_R using values of A and I as follows:

Solid unit or filled-cell hollow unit pilasters:

$$A = bT;\ I = \frac{bT^3}{12} + bT(\frac{x}{2})^2 = \frac{bT}{12}(T^2 + 3x^2)$$

Hollow unit pilasters:

$$A = 0.5bT;\ I = 0.6\frac{bT^3}{12} + 0.5bT(\frac{x}{2})^2 = \frac{bT}{12}(0.6T^2 + 1.5x^2)$$

Then; M_R should $\geq M_E$ for lateral stability where:

M_E = moment due to earth pressure or other lateral loads to be carried by the pilasters.

Check: Calculate the extreme fiber stresses, (f) in the pilaster from $f = P/A \pm Mc/I$ as follows:

$$f_1 = \frac{P+W_D}{A} + \frac{[P(x'-x/2) + W_D(x/2) - M_E]c}{I};\quad f_2 = \frac{P+W_D}{A} - \frac{[P(x'-x/2) + W_D(x/2) - M_E]c'}{I}$$

Then f should be $\leq$ allowable compressive or tensile stress in plain masonry.

Sample Calculation

Assume: Pilasters of filled-cell hollow unit construction and that all of the earth pressure load on the walls will be transmitted horizontally to the pilasters. Take $f_t = 0$ and, $a = 12$ ft.; $h = 7$ ft.; $H = 8$ ft.; $w = 25$ lb./cu.ft.; $t = 11.62$ in.; $T = 19.62$ in.; $P = 10000$ lb.; $P_D = 8000$ lb.; unit weight (pilaster) = 140 lb./cu.ft.; $x' = 10$ in., $b = 15.62$ in.

Compute: $x = \frac{19.62}{2} - \frac{11.62}{2} = 4$ in.; $\frac{x}{2} = 2$ in.; $c = \frac{19.62}{2} + 2 = 11.81$ in.; $c' = 7.81$ in.

$A = (15.62)(19.62) = 306$ sq.in.; $I = \frac{306}{12}[(19.62)^2 + 3(4)^2] = 11{,}000\ in.^4$

$W_D = \frac{306}{144}(4)(140) = 1200$ lb.; $S = \frac{11{,}000}{11.81} = 930\ in.^3$

$M_R = [0 + \frac{(8000+1200)}{306}](930) + (8000)(10-2) + (1200)(2) = 94{,}300$ in.lb.

From Fig. 7 with h = 7ft., H = 8ft., and $w = 25$ lb./cu.ft.: $M_E = 7550$ in.lb./lin. ft. of wall.

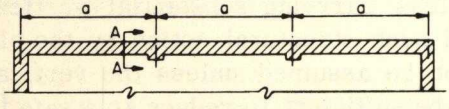
Since a = 12 ft., $M_E = 7550(12) = 90{,}600$ in.lb./pilaster

$M_R = 94{,}300$ in.lb. $> M_E = 90{,}600$ in.lb.

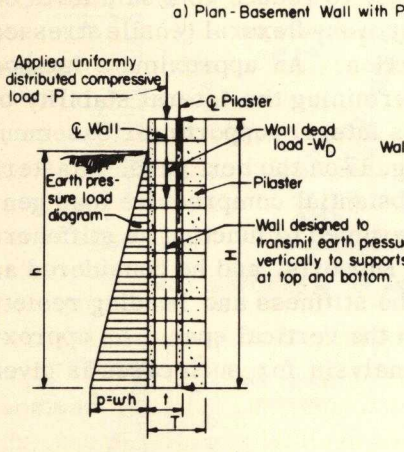
Check: $f_1 = \frac{10000+1200}{306} + \frac{[(10000)(8) + 1200(2) - 90{,}600]11.81}{11{,}000} = 36.7 - 8.8 = 27.9$ psi comp.

$f_2 = 36.7 + 8.8 = 45.5$ psi comp.

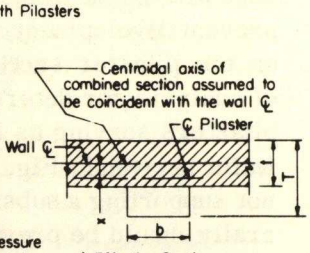
Figure 17 - Approximate Analysis of Pilaster Serving as a Lateral Support for Basement Walls.



a) Plan - Basement Wall with Pilasters



b) Load Diagram - Section A-A



c) Effective Section Assumed in Analysis

Approximate Lateral Stability Analysis

Referring to sketches b) and c):

Calculate the dead load resisting moment, M_R, of the combined wall-pilaster section as follows (assuming P acts along ℄ wall):

$M_R = (f_t + \frac{P_D + W_D}{A})S$; if P is eccentric with respect to ℄ wall, its effect on M_R may be included as shown in Fig. 17.

f_t = allowable tensile stress in plain masonry, psi.
P_D = dead load component of applied vertical compressive load, lb./lin.ft.
W_D = weight of wall above point of maximum moment, lb./lin. ft.
A = net area of the effective section, sq.in./lin.ft.
S = section modulus of section, $in.^3$/lin.ft.

For purposes of approximate analysis, values for A and S in the above formula may be calculated as follows:

With walls and pilasters built of hollow units and with face shell mortar bedding:

$A = 4t$

$$S = \frac{I}{1/2} = \frac{4t^2}{3} + \frac{bT}{6at}[0.6T^2 + 6x^2]$$

With walls and pilasters built of solid units and full mortar bedding or of filled-cell hollow unit construction:

$A = 12t$

$$S = \frac{I}{1/2} = 2t^2 + \frac{bT}{6at}[T^2 + 12x^2]$$

With walls built of hollow units and face shell mortar bedding and with pilasters built of solid units and full mortar bedding or of filled-cell hollow unit construction:

$A = 4t$

$$S = \frac{I}{1/2} = \frac{4t^2}{3} + \frac{bT}{6at}[T^2 + 12x^2]$$

In the above t, T, b, and x represent the actual wall thickness, pilaster thickness and width, and the distance between the wall and pilaster centerlines respectively expressed in inches, and a the wall length between pilasters expressed in feet.

Sample Calculation

Assume: Wall and pilasters of hollow unit construction with face shell mortar bedding and:
$t = 11.62$ in.; $T = 19.62$ in.; $b = 15.62$ in.; $a = 12$ ft.; $H = 8$ ft.; $h = 7$ ft.; $P = 1500$ lb.; $P_D = 1000$ lb.; $w = 25$ lb./cu.ft.; unit weight (wall) = 60 lb./cu.ft.; $f_t = 12$ psi.

Compute: $x = \frac{19.62}{2} - \frac{11.62}{2} = 4$ in.; $W_D = 60(4) = 240$ lb./ft.

$A = 4(11.62) = 46.5$ sq.in./ft.

$S = \frac{4(11.62)^2}{3} + \frac{(15.62)(19.62)}{6(12)(11.62)}[0.6(19.62)^2 + 6(4)^2]$

$= 181 + 0.367(230 + 96) = 300\ in.^3$/ft.

$M_R = (12 + \frac{1000+240}{46.5})300 = 11{,}600$ in.lb./ft.

From Fig. 7 with H = 8 ft. and h = 7 ft.:
$M_E = 7550$ in.lb./ft.

$M_R = 11{,}600 > M_E = 7550$ in.lb./ft.

∴ Wall will have sufficient lateral stability.

Figure 18 - Approximate Analysis of Pilaster Serving to Stiffen Basement Walls in the Vertical Span.

6. Bond Beams and Joint Reinforcement

The flexural strength and stiffness of concrete masonry walls in the horizontal span can often be increased or made more reliable through provision of horizontal reinforcement in the walls, either in the form of bond beams or joint reinforcement. Such measures are sometimes used where the lateral load to be carried will exceed that which can safely be resisted by unreinforced walls of practical and economical thickness, but not by a sufficiently large margin to justify a fully reinforced design. They are also recommended and used in walls laid in stacked or other bond patterns producing continuous vertical joints in the walls, since such walls normally will have reduced flexural strengths in the horizontal span as compared to walls laid in running bond. In the majority of cases, however, bond beams and joint reinforcement are used primarily for crack control purposes and to preserve rather than increase the flexural strength of walls in the horizontal span in the event that vertical cracking due to shrinkage or other causes should develop and impair the strength of the masonry itself.

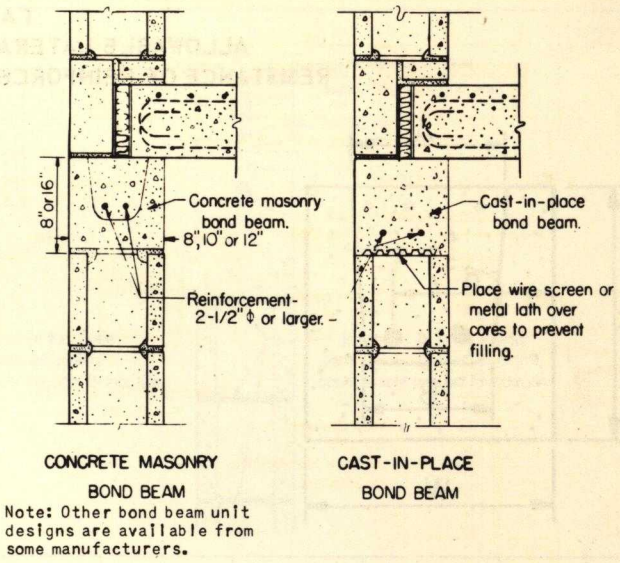
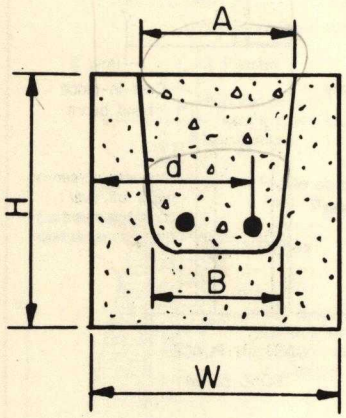

8" or 16"
Concrete masonry bond beam. 8", 10" or 12"
Reinforcement - 2-1/2" ϕ or larger.
Cast-in-place bond beam.
Place wire screen or metal lath over cores to prevent filling.
CONCRETE MASONRY BOND BEAM
CAST-IN-PLACE BOND BEAM
Note: Other bond beam unit designs are available from some manufacturers.

Figure 19 - Types and Suggested Location of Bond Beams in Concrete Masonry Foundation Walls.
(Note: The concrete masonry type is recommended as being more economical and satisfactory.)

Typical bond beam construction, as illustrated in Fig. 19, may be either of the concrete masonry or the cast-in-place type; however, the concrete masonry type is generally recommended as a more economical and satisfactory construction for most applications. In foundation walls, bond beams are most advantageously located at or near the top of the wall and built so as to extend continuously around the foundation. Where used in this manner, they act not only as edge stiffening beams, increasing the rigidity and flexural strength of the walls in the horizontal span, but also serve to distribute concentrated loads over the wall length and thickness, eliminate the need for separate lintels above openings where the head of the opening falls at the bottom of the bond beam course, and prevent or minimize vertical crack development. Since bond beams are, essentially, reinforced concrete members, they may be designed in the same manner as reinforced concrete beams. The provision of bond beams in a foundation wall may add appreciably to the cost of the wall construction, and careful consideration should be given to the need for such measures in each design to insure that the loads and stresses or the importance of the construction is such as to justify their use. Data on the lateral flexural and shearing resistance of reinforced concrete masonry bond beams are shown in Table 11 on the next page.

Horizontal joint reinforcement serves much the same purpose as bond beams in foundation walls. It is helpful, primarily, in minimizing the width of any vertical cracks which develop and in preserving, to a certain extent, the horizontal flexural strength of the walls after cracking. As in the case of bond beams, it is most effective when used in the upper portions of the wall height, as shown in Fig. 20, page 27, and placed so as to extend continuously around the walls. Where window openings extend to the top of the wall, the most effective location will be in the mortar joints immediately below the openings. Various types of prefabricated joint reinforcement, similar to those shown in Fig. 20, are available in most areas, and their use is recommended over single wire or pencil rod reinforcement. As a general rule, the diameter of the longitudinal wire embedded in the mortar joint should not exceed No. 6 gage if a 3/8 in. joint thickness is to be maintained; 3/16 in. dia., No. 8, or No. 9 gage deformed wire of high tensile strength steel is commonly used in the prefabricated types.

TABLE 11.
ALLOWABLE LATERAL FLEXURAL AND SHEARING RESISTANCE OF REINFORCED CONCRETE MASONRY BOND BEAMS



DESIGN FACTORS:

$f'_c = 2500$ psi; $f_s = 20000$ psi

$f'_m = 2000$ psi

$f_m = 0.33\, f'_m = 667$ psi

$v = 0.02\, f'_m = 40$ psi

$u = 160$ psi

$E_m = 1000\, f'_m = 2{,}000{,}000$ psi

$n = \frac{E_s}{E_m} = 15$

Assumed reinforcing cover = 1/2 in.
Only one reinforcing bar used in computing resisting moment.

H in.	W in.	A in.	B in.	d in.	Reinforcing	Maximum Allowable Resisting Moment, in lb. *	Maximum Allowable Shear, lb. *
7-5/8	7-5/8	4-1/2	4	5	2-No. 4	18,100	1130 ‡
					2-No. 5	21,300 †	1330 §
					2-No. 6	23,300 †	1290 §
	9-5/8	6-1/2	6	7	2-No. 4	25,600	1610 ‡
					2-No. 5	37,500 †	1890 §
					2-No. 6	41,300 †	1850 §
	11-5/8	8-1/2	8	9	2-No. 4	33,400	2100 ‡
					2-No. 5	50,000	2460 §
					2-No. 6	63,300 †	2400 §
15-5/8	7-5/8	4	3-1/4	4-5/8	2-No. 4	17,200	1000 ‡
					2-No. 5	25,700	1300 ‡
					2-No. 6	33,200 †	1510 ‡
	9-5/8	6	5-1/4	6-5/8	2-No. 4	24,800	1560 ‡
					2-No. 5	37,500	1900 ‡
					2-No. 6	51,900	2230 ‡
	11-5/8	8	7-1/4	8-5/8	2-No. 4	32,500	2040 ‡
					2-No. 5	49,300	2490 ‡
					2-No. 6	68,600	2940 ‡

* These values may be increased by one-third when designing for wind, blast, or earthquake forces in combination with dead and other live loads.

† Indicates maximum resisting moment is limited by compressive stress in masonry.

‡ Indicates maximum load is limited by bond.

§ Indicates maximum load is limited by shear.

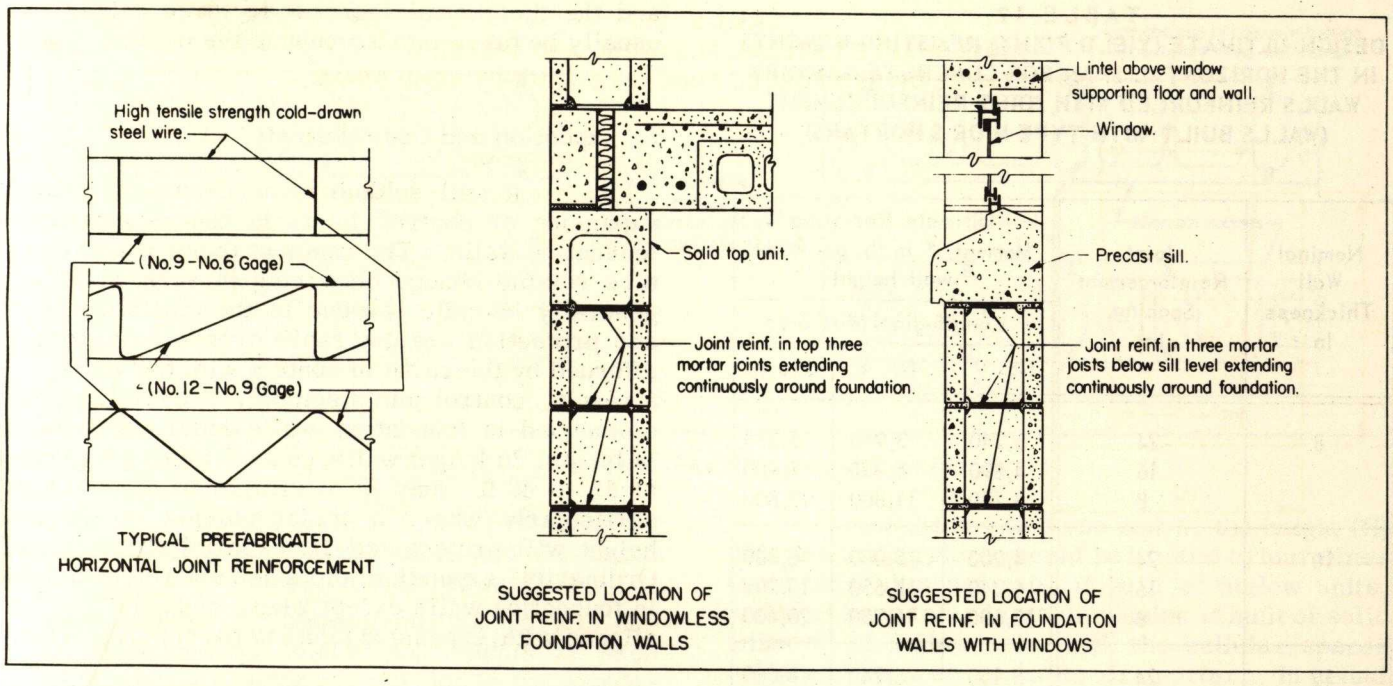

High tensile strength cold-drawn steel wire.
(No. 9 - No. 6 Gage)
(No. 12 - No. 9 Gage)
TYPICAL PREFABRICATED HORIZONTAL JOINT REINFORCEMENT
Solid top unit.
Joint reinf. in top three mortar joints extending continuously around foundation.
SUGGESTED LOCATION OF JOINT REINF. IN WINDOWLESS FOUNDATION WALLS
Lintel above window supporting floor and wall.
Window.
Precast sill.
Joint reinf. in three mortar joists below sill level extending continuously around foundation.
SUGGESTED LOCATION OF JOINT REINF. IN FOUNDATION WALLS WITH WINDOWS

Figure 20 - Types and Suggested Location of Horizontal Joint Reinforcement in Concrete Masonry Foundation Walls.

Referring to Fig. 21, the approximate ultimate resisting moment of a cracked section reinforced with joint reinforcing may be calculated from the following formula:

$$M_s = \frac{7}{8} A_s f_y d, \text{ where,}$$

A_s = effective cross-sectional area of wire embedded in mortar on tension side of the section, sq. in.

f_y = yield point strength of wire, psi (64,000 psi for cold-drawn steel wire)

d = depth of section from center of tension wire to compression face of section, in.

To develop its yield point resistance, the wire must be strongly bonded to the mortar and the mortar must be well bonded to the masonry units. A deformed type of wire and a strong mortar, as specified elsewhere, are recommended.

Calculated ultimate resisting moments for 8, 10, and 12 in. thick walls containing various amounts of joint reinforcement are given in Table 12 on the next page. The values are based on the assumption of a cracked section and on the minimum yield point strength of cold-drawn steel wire (64,000 psi), and are conservative with respect to the actual ultimate moment which would be required to cause actual collapse or total failure. When comparing the calculated values given in Table 12 with actual or calculated ultimate resisting moments for unreinforced walls, as given in Table 6, page 17, recognition should be given to the inherent reserve strength and the greater reliability of reinforced as compared to unreinforced walls. This may be accomplished by varying the safety factor employed in arriving at design resisting moment values for the respective wall constructions. For example, the values given in Table 12 may be divided by two to arrive at a design resisting moment for a reinforced wall whereas the design resisting moment for an unreinforced wall may be taken as one-quarter of the ultimate values given in Table 6, or some lesser or greater value depending upon the strength of the units and the probability of vertical cracking developing in and impairing the horizontal flexural resistance of the unreinforced wall.

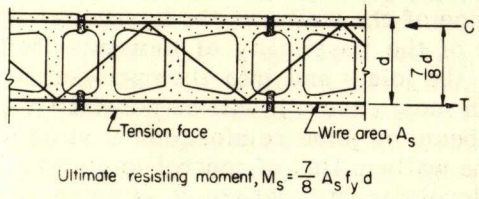


Figure 21 - Horizontal Section at Mortar Joint Containing Embedded Joint Reinforcement.

Where it is desired to employ stacked bond construction in basement walls, it is recommended that continuous horizontal reinforcement, sufficient to provide a horizontal flexural resistance at least equal to that which would be obtained with running bond, be incorporated in the wall construction. As a general rule, bond beams reinforced with 2 No. 5 bars (one near each face) and spaced not more than 42 in. o.c., or joint re-

TABLE 12.
DESIGN ULTIMATE (YIELD POINT) RESISTING MOMENTS IN THE HORIZONTAL SPAN FOR CONCRETE MASONRY WALLS REINFORCED WITH JOINT REINFORCEMENT (WALLS BUILT WITH TYPE M OR S MORTARS)

Nominal Wall Thickness In.	Joint Reinforcement Spacing, In.	Ultimate Resisting Moment, * in lb. per ft. of wall height		
		Longitudinal Wire Size		
		No. 9	No. 8	3/16 in.
8	24	3,240	3,940	5,270
	16	4,890	5,920	7,900
	8	9,740	11,800	15,800
10	24	4,200	5,080	6,800
	16	6,310	7,650	10,200
	8	12,600	15,250	20,400
12	24	5,150	6,240	8,360
	16	7,750	9,400	12,550
	8	15,450	18,750	25,100

* Assuming use of prefabricated joint reinforcement made from cold-drawn steel wire and with parallel side rods spaced 6 in., 8 in. and 10 in. apart for the 8 in., 10 in. and 12 in. walls respectively. A yield point strength of 64,000 psi and effective depths of 6.82 in., 8.82 in. and 10.82 in. were assumed in the calculations. Resisting moments based on ultimate strength of wire (80,000 to 100,000 psi) will exceed the values shown by 25 to 56 percent.

inforcement with No. 9 gage or larger side wires in each horizontal mortar joint 8 in. o.c., will be adequate.

When considering the use of bond beams and joint reinforcement in foundation walls, it is important to remember that neither of these measures will add appreciably to the strength of the walls in the vertical span. Also, the increase in horizontal flexural strength which can be obtained through use of joint reinforcement is limited, due to practical limitations on the amount of steel which can be embedded in the joints and the bond strength which can be developed between the mortar, the reinforcement, and the wall units. Generally speaking, it will be prudent to neglect any theoretical increase in horizontal flexural strength which may be obtained with joint reinforcement and to base the design of walls on the same wall height, depth below grade and lateral support requirements recommended for unreinforced walls. Bond beams, on the other hand, may be so designed as to contribute appreciably to wall strength and stiffness and the theoretical increase in these values may usually be taken into account in the design.

7. Expansion and Control Joints.

It will seldom be necessary to provide expansion or control joints in concrete masonry foundation walls. The range of thermal and moisture volume change occurring in such walls will generally be quite low due to the insulating effect and protection against rapid or excessive drying afforded by the earth in contact with the walls. As a result, control joints seldom, if ever, are recommended in foundation walls under about 100 ft. in length. In longer walls, control joints at spacings of 35 to 40 ft. may be desirable in some cases, particularly where a major portion of the wall height will project and be exposed above grade. Ordinarily, expansion joints are not recommended in foundation walls except where required by, and aligned with, expansion joints in the superstructure.

Where foundation walls serve only as supporting walls, do not enclose basement areas and are required to resist little or no lateral load, the provision of control or expansion joints presents few problems or difficulties. The joints should be of the tongue and groove type or otherwise keyed to prevent faulting at the joints as in the suggested details shown in Fig. 22. They should extend from the top of the foundation wall down to the top of the footing, except that in long walls supporting wood frame construction, it usually will be desirable to provide a reinforced concrete masonry bond beam at or near the top of the wall and continuous across the control joints to minimize the possibility of excessive movements developing at the control joints and causing cracking of plaster or other interior finishes in the superstructure. Also, where control or expansion joints will be built into foundation walls, reinforcement should be provided in the wall footing below such joints to insure that no vertical faulting or differential settlement will develop should movement at the joints cause cracks to form in the footing.

It is advisable to avoid placing control joints in basement walls wherever possible, due to the fact that they tend to reduce the flexural resistance of the walls in the horizontal span and because of the possibility of moisture infiltrating through the joints and into the enclosed basement area. In most cases, it will be possible to provide a bond beam or joint reinforcement at or near the top of the walls in lieu of control joints to minimize crack development. Where it is imperative that expansion or control joints be used, special attention should be given to the design of the joints to

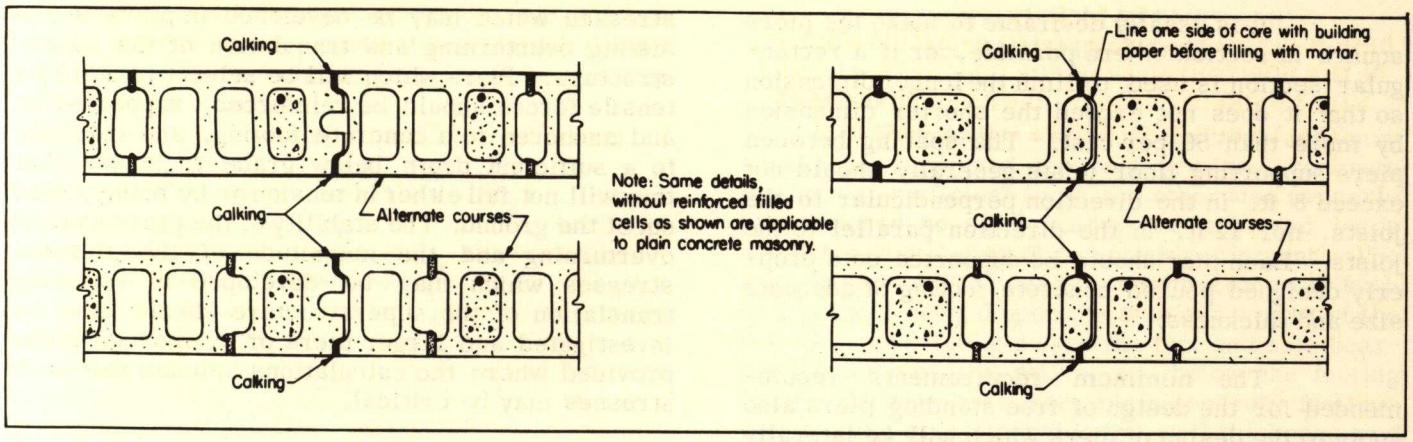


Figure 22 - Typical Shear-Resisting Control Joint Details.

insure, insofar as possible, adequate provision for transfer of shear across the joints and maximum protection against moisture infiltration.

Where the above grade masonry is designed with control joints, movements at such joints may cause cracking to develop in the foundation walls below. To minimize this possibility, it is suggested in such cases that a continuous bond beam be provided at the top of the foundation walls and that the control joints in the above grade walls be stopped at this bond beam.

8. Pier Foundations

In some areas where crawl space type dwellings are popular, pier type foundations are often used in lieu of wall type foundations to support small frame and masonry veneer buildings not over two stories in height. Such construction is most extensively used in warm, humid climates where basements offer fewer advantages than in colder climates.

The design of pier type foundations will vary with different building designs, loads and other factors. Generally speaking, they may be either of the free standing pier or of the pier and curtain wall type as shown in Fig. 23. In the free standing pier type, each pier is designed and built to function as an independent element in supporting and transmitting the building loads to the ground. Foundations of this type are usually recommended only for one story frame buildings. In the pier and curtain wall type, the individual piers forming the perimeter of the foundation are connected together by means of non-load bearing curtain walls extending continuously between and bonded into the piers and supported on a concrete footing poured integrally with the pier footings. The curtain walls serve to provide some measure of lateral support for the piers, and also to enclose the crawl space under the structure. Foundations of this type are used to support frame construction not over two stories in height, and masonry veneered frame construction not over one and one-half stories in height.

According to some codes, the height (H) of free standing piers should be limited to four times their least dimension (t) if built of hollow units, and ten times their least dimension if built of solid units or of hollow units with the cellular spaces solidly filled with concrete or mortar. In actual practice an $\frac{H}{t}$ ratio as great as ten for unreinforced foundation piers subject to lateral loads would be regarded as dangerous design. Similarly, the minimum pier section permitted in some codes, 8 x 12 in. for piers built of solid units and 8 x 16 in. for hollow unit piers, should not be taken as adequate under all conditions.

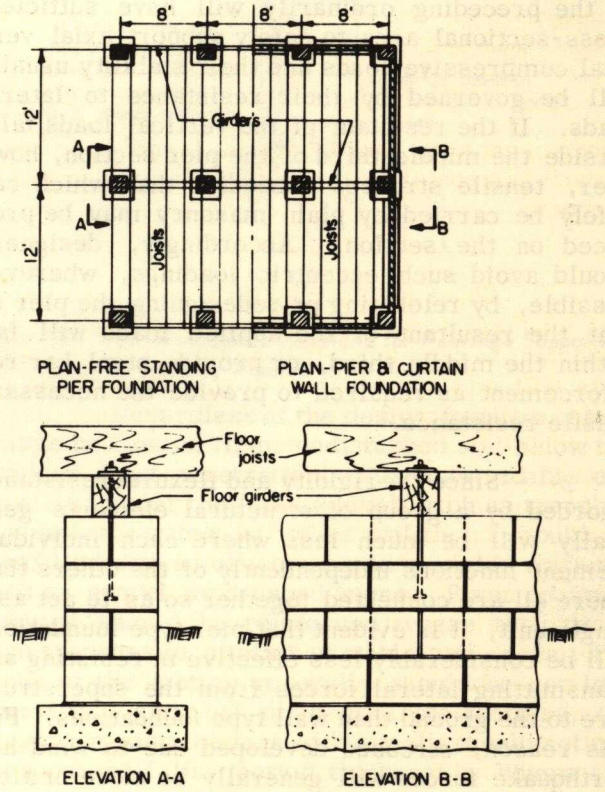


Figure 23 - Pier Foundations.

It is usually desirable to make the piers square in section where possible, or if a rectangular section is used, to limit the longer dimension so that it does not exceed the shorter dimension by more than 50 per cent. The spacing between piers supporting floor joists generally should not exceed 8 ft. in the direction perpendicular to the joists, nor 12 ft. in the direction parallel to the joists. Each pier should be supported on a properly designed poured concrete footing of adequate size and thickness.

The minimum requirements recommended for the design of free standing piers also apply to the design of piers which will be laterally supported by curtain walls. In such construction, the curtain walls should be not less than 4 in. in nominal thickness and the height limited to a maximum of twenty times the wall thickness if built of solid units and eighteen times the wall thickness if built of hollow units. The difference between ground levels on the exterior and interior side of 4 in. curtain walls generally should not exceed about 2 ft.

The same allowable working stresses as recommended for the design of concrete masonry walls may also be used for the design of concrete masonry piers. Somewhat higher values in compression, 105 psi and 95 psi on the gross section, are permitted for piers of filled cell hollow unit construction laid up with Type M or Type S mortar respectively. Piers proportioned as recommended in the preceding ordinarily will have sufficient cross-sectional area to safely support axial vertical compressive loads and their stability usually will be governed by their resistance to lateral loads. If the resultant of the vertical loads falls outside the middle third of the pier section, however, tensile stresses exceeding that which can safely be carried by plain masonry may be produced on the section. Accordingly, designers should avoid such eccentric loadings, wherever possible, by relocating or redesigning the pier so that the resultant of the applied loads will fall within the middle third, or provide steel bar reinforcement as required to provide the necessary tensile resistance.

Since the rigidity and flexural resistance afforded by a group of structural elements generally will be much less where each individual element functions independently of the others than where all are connected together so as to act as a single unit, it is evident that pier type foundations will be considerably less effective in resisting and transmitting lateral forces from the superstructure to the ground than wall type foundations. For this reason, stresses developed due to wind and earthquake forces will generally be more critical in pier type than in wall type foundations. Consequently, careful attention should be given to the stresses which may be developed in piers in resisting overturning and translation of the superstructure. Piers which will be subject to uplift or tensile forces should be reinforced, supported on and anchored to a concrete footing, and extended to a sufficient depth below grade to insure that they will not fail either in tension or by being pulled out of the ground. The stability of the piers against overturning and the magnitude of the flexural stresses which may be developed in resisting translation of the superstructure should also be investigated and larger piers or reinforced piers provided where the calculations indicate that such stresses may be critical.

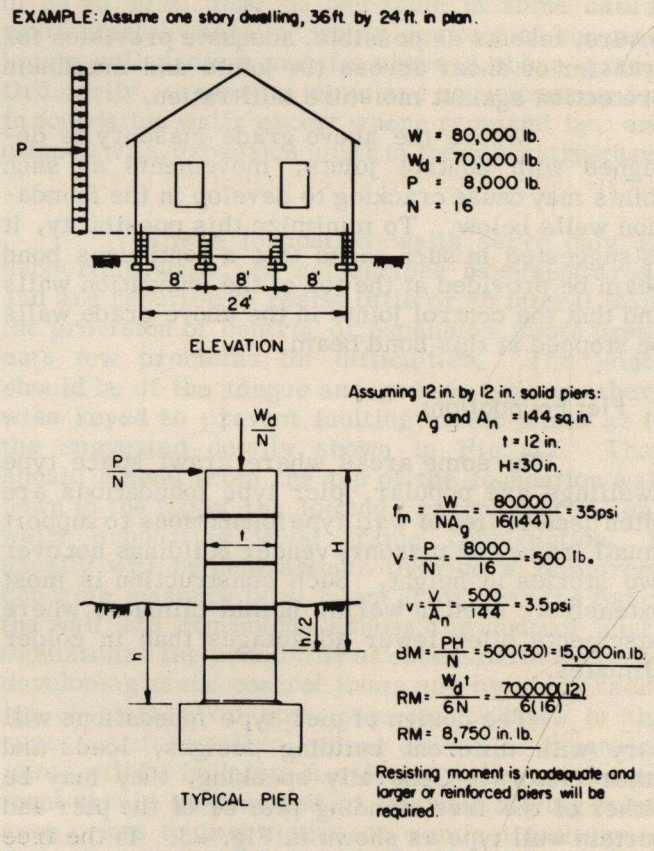

EXAMPLE: Assume one story dwelling, 36 ft. by 24 ft. in plan.
P
W = 80,000 lb.
W_d = 70,000 lb.
P = 8,000 lb.
N = 16
8'
8'
8'
24'
ELEVATION
$\frac{W_d}{N}$
$\frac{P}{N}$
t
H
h/2
h
TYPICAL PIER
Assuming 12 in. by 12 in. solid piers:
A_g and A_n = 144 sq. in.
t = 12 in.
H = 30 in.
$f_m = \frac{W}{NA_g} = \frac{80000}{16(144)} = 35$ psi
$V = \frac{P}{N} = \frac{8000}{16} = 500$ lb.
$v = \frac{V}{A_n} = \frac{500}{144} = 3.5$ psi
$BM = \frac{PH}{N} = 500(30) = 15{,}000$ in. lb.
$RM = \frac{W_d t}{6N} = \frac{70000(12)}{6(16)}$
RM = 8,750 in. lb.
Resisting moment is inadequate and larger or reinforced piers will be required.

Figure 24 - Analysis of a Pier Foundation for a Small Building.

Referring to Fig. 24, pier stresses and required sections can be calculated by the following approximate method, where,

W = total dead and live load, lb.

W_d = total dead load, lb.

P = total wind load (or other lateral force), lb.

N = number of piers

A_g and A_n = respectively, gross and net cross-sectional area of one pier, sq. in.

t = width of pier in direction of force P, in.

H = height of pier, top to mid-point below ground, in.

Assuming W and P are uniformly distributed to all piers, and that A_g, A_n, t and H are similar for all piers, stresses and moments in each pier may be calculated as follows:

$$f_m \text{ (compressive unit stress, psi)} = \frac{W}{NA_g}$$

$$V \text{ (total shear, lb.)} = \frac{P}{N}$$

$$v \text{ (shearing unit stress, psi)} = \frac{V}{A_n}$$

$$BM \text{ (bending moment, in lb.)} = \frac{PH}{N}$$

$$RM \text{ (dead load resisting moment, in. lb.)} = \frac{W_d t}{6N}$$

Values of f_m and v should not exceed the allowable stresses for the type of construction used. The resisting moment due to dead load should at least equal the bending moment if the piers are not reinforced and tension is to be avoided. For the case shown in Fig. 24, the analysis indicates that the 12 x 12 in. pier section would be inadequate unless reinforced or increased in size.

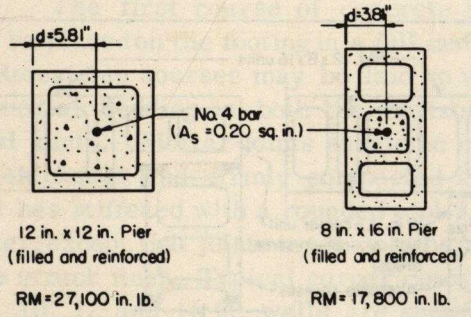


Figure 25 - Typical Reinforced Pier Sections.

Referring to Fig. 25 the resisting moment of a 12 x 12 in. pier built of hollow pier block with the core filled with concrete and reinforced with one No. 4 bar would be: $RM = \frac{7}{8} A_s f_s d = (0.875)(0.20)(26,667)(5.81) = 27,100$ in. lb. Thus, neglecting the dead load resisting moment, the reinforced design is more than adequate. An 8 x 16 in. hollow concrete block pier, reinforced with one No. 4 bar, would develop an internal resisting moment of 17,800 in. lb. which also exceeds the bending moment. Overturning moment, which would have the effect of increasing the stress in some piers and reducing it in others, is not critical and has been neglected in the above case.

More exact methods of analysis, which would produce more favorable results for the unreinforced piers, are seldom employed. In general, when H exceeds about 24 in. a reinforced design will be more reliable and only slightly more costly than the plain masonry type.

In areas subject to earthquakes, pier type foundations, if used, should be specially designed and in all cases properly reinforced.

B. CONSTRUCTION DETAILS AND RECOMMENDATIONS

1. Footings

Foundation walls of concrete masonry should be supported on cast-in-place concrete footings of sufficient width and otherwise properly proportioned to safely carry the expected loads without excessive or differential settlement. The size, design and the need for reinforcement in such footings will depend upon the size and weight of the building supported and upon the type and safe bearing capacity of the soil at and below the footing level. As a general rule, plain concrete footings with a width equal to twice the wall thickness and a depth equal to the wall thickness, as shown in Fig. 26, will be adequate for average soil conditions for foundations supporting dwellings and similar small buildings. Where footings are to be supported on low bearing capacity soils, such as soft clay or silt, or where the loads to be supported will exceed those normally encountered in residential construction, larger footing widths and depths may be required. Similarly, if the soil under the footing varies appreciably in compressibility, it will generally be advisable to provide some longitudinal reinforcement in the footing in the event that differential settlement sufficient to cause failure and cracking in the footing should develop. Typical safe bearing values for various soils are shown in Table 13, page 33.

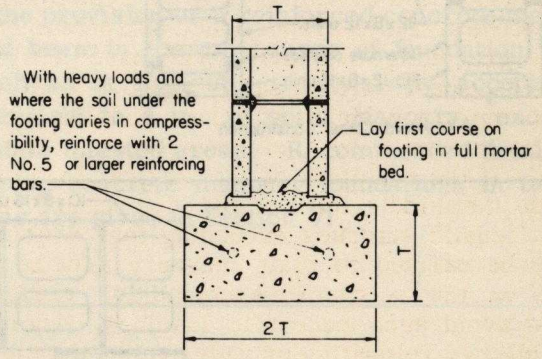

T
With heavy loads and where the soil under the footing varies in compressibility, reinforce with 2 No. 5 or larger reinforcing bars.
Lay first course on footing in full mortar bed.
T
2T

Figure 26 - Recommended Dimensions for Footings Supporting Plain Concrete Masonry Foundation Walls.

Regardless of the design, footings should always be cast on firm, undisturbed soil below the level of frost penetration. Any unavoidable excavations under the footing level, such as trenches for sewer, water or other utilities, should be backfilled with a well compacted granular material such as gravel or crushed stone. Plain concrete footings should be reinforced where they cross such trenches or similar unstable soil areas. The depth of the footing generally should be not less than 8 in., and with plain concrete footings, the projection past the face of the wall should not exceed one-half the footing thickness. Where tile drains are installed around the perimeter of footings, care should be taken to insure that the drain

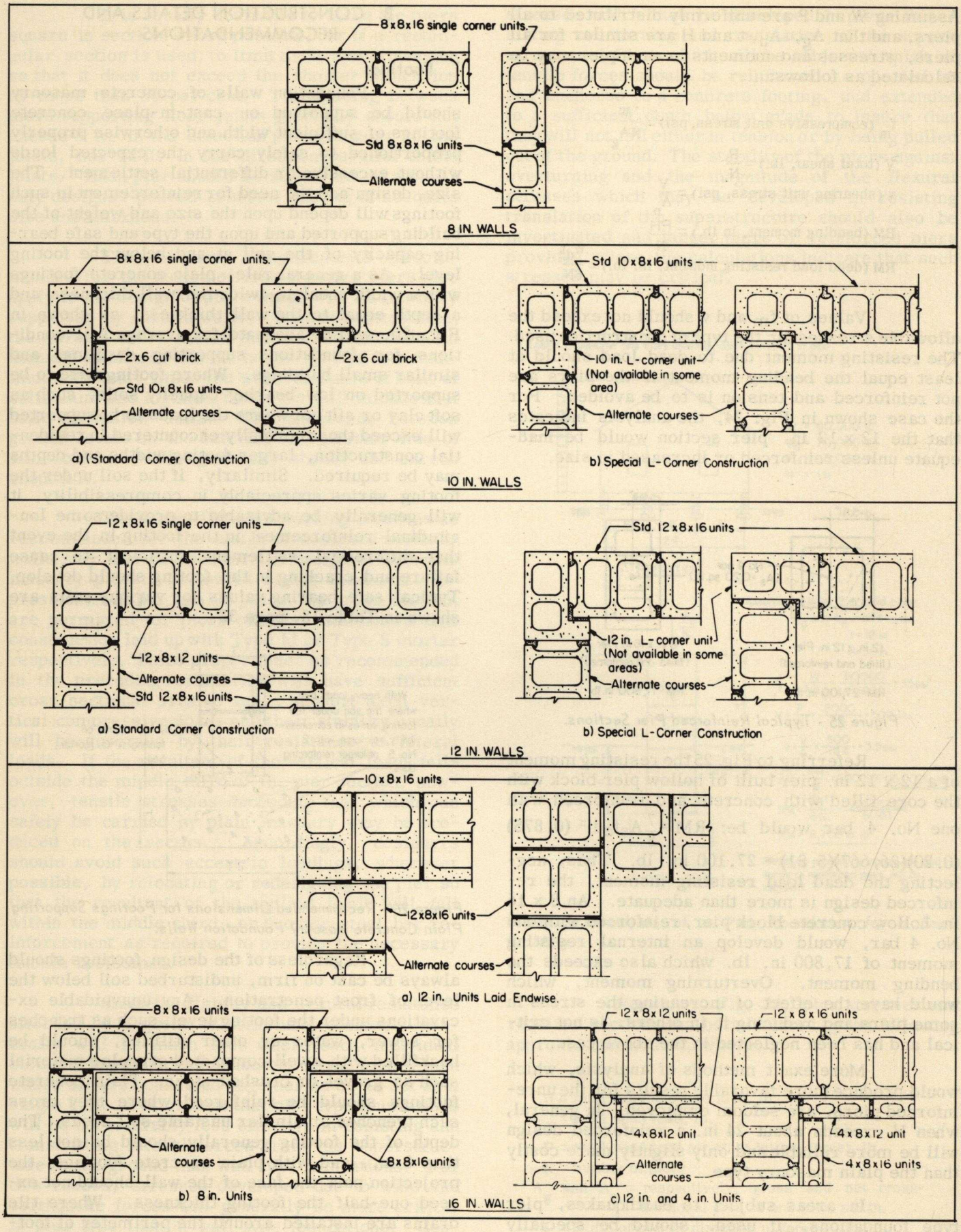

8 x 8 x 16 single corner units.
Std 8 x 8 x 16 units
Alternate courses
8 IN. WALLS
8 x 8 x 16 single corner units.
2 x 6 cut brick
2 x 6 cut brick
Std 10 x 8 x 16 units
Alternate courses
a) (Standard Corner Construction
Std 10 x 8 x 16 units
10 in. L-corner unit
(Not available in some area)
Alternate courses
b) Special L-Corner Construction
10 IN. WALLS
12 x 8 x 16 single corner units
12 x 8 x 12 units
Alternate courses
Std 12 x 8 x 16 units
a) Standard Corner Construction
Std. 12 x 8 x 16 units
12 in. L-corner unit
(Not available in some areas)
Alternate courses
b) Special L-Corner Construction
12 IN. WALLS
10 x 8 x 16 units
12 x 8 x 16 units
Alternate courses
a) 12 in. Units Laid Endwise.
8 x 8 x 16 units
Alternate courses
8 x 8 x 16 units
b) 8 in. Units
12 x 8 x 12 units
12 x 8 x 16 units
4 x 8 x 12 unit
4 x 8 x 12 unit
Alternate courses
4 x 8 x 16 units
c) 12 in. and 4 in. Units
16 IN. WALLS

Figure 27 - Typical Corner Construction for 8, 10, 12, and 16 in. Walls.

invert is kept well above the level of the base of the footing to avoid any possibility of washing out, undermining, or otherwise disturbing the soil under the footing. In the event that load, soil or other considerations require the use of relatively wide footings of shallow depth, a reinforced design involving transverse as well as longitudinal reinforcement will generally be required.

The design of footings to support heavy loads or for unusual soil conditions involves many factors and variables which are beyond the scope of this information. Where such special conditions are encountered, reference should be made to the information and recommendations on the design of both plain and reinforced concrete footings given in most textbooks on concrete design.

TABLE 13.
ALLOWABLE BEARING VALUE OF SOILS

Type of Soil	Safe Bearing Capacity, tons per sq. ft.
Soft clay; soft broken shale.	1
Fine, loose sand; stiff clay.	2
Coarse, loose sand; fine compact sand; loose sand-gravel mixtures.	3
Coarse, compact sand; loose gravel; soft shales; hard dry clay.	4
Compact gravel and sand-gravel mixtures.	6
Compacted, partially cemented gravels or sands; hardpan.	10

2. Wall Construction

The first course of concrete masonry should be placed on the footing in a full bed of mortar. Remaining courses may be laid up with face shell mortar bedding in both the horizontal and vertical joints. Mortar joints should be made 3/8 in. in thickness and firmly compacted after the mortar has stiffened with a rounded tool 5/8 in. in diameter, except that joints in areas to be plastered may be struck flush. Typical corner constructions with 8, 10, 12 and 16 in. walls are shown in Fig. 27.

To insure adequate distribution of concentrated loads from joists, beams, or other members, the bearing course in hollow unit walls should be built of solid masonry or solid concrete. Most regulations require a solid masonry bearing course 4 in. thick under floor slabs, joists, beams, and girders. The use of solid top units, filling of the cores of hollow units with concrete or mortar, or the provision of a reinforced concrete masonry bond beam in the top course of foundation walls, as shown in Fig. 28, is generally accepted as equivalent to 4 in. of solid masonry, except in termite infested areas. Recommended details for capping concrete masonry foundations in termite areas are given in Section VI.

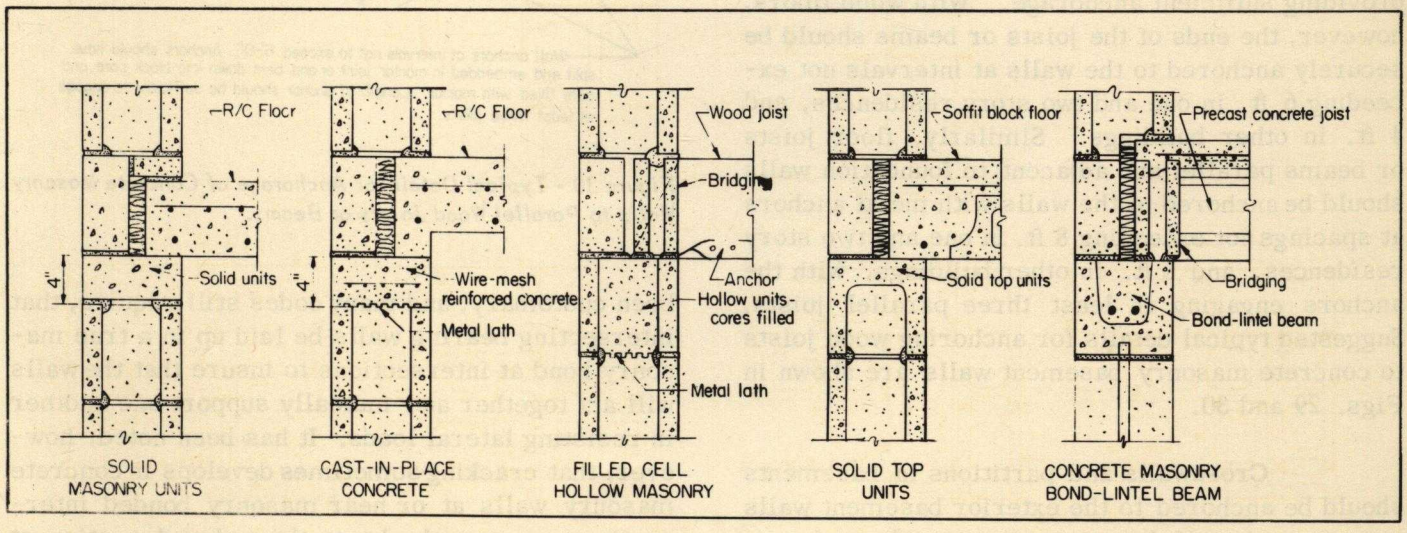


Figure 28 - Typical Methods of Constructing Course Supporting Floors in Concrete Masonry Foundation Walls.

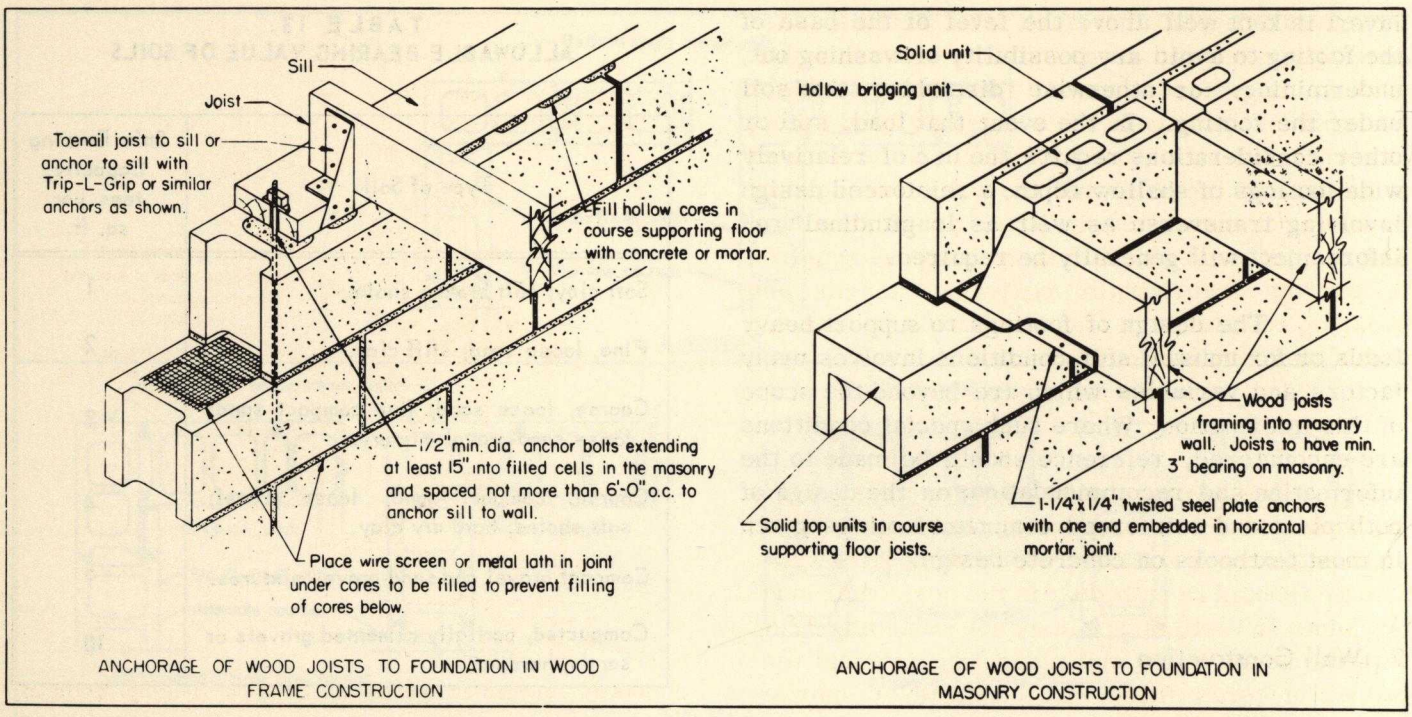

Sill
Joist
Toenail joist to sill or anchor to sill with Trip-L-Grip or similar anchors as shown.
Fill hollow cores in course supporting floor with concrete or mortar.
1/2" min. dia. anchor bolts extending at least 15" into filled cells in the masonry and spaced not more than 6'-0" o.c. to anchor sill to wall.
Place wire screen or metal lath in joint under cores to be filled to prevent filling of cores below.
ANCHORAGE OF WOOD JOISTS TO FOUNDATION IN WOOD FRAME CONSTRUCTION
Solid unit
Hollow bridging unit
Wood joists framing into masonry wall. Joists to have min. 3" bearing on masonry.
1-1/4"x1/4" twisted steel plate anchors with one end embedded in horizontal mortar joint.
Solid top units in course supporting floor joists.
ANCHORAGE OF WOOD JOISTS TO FOUNDATION IN MASONRY CONSTRUCTION

Figure 29 - Suggested Methods of Anchoring Wood Joists Bearing on Concrete Masonry Foundation Walls.

3. Anchorage

Where basement walls are to be laterally supported at the top by the first floor, the need for special anchorage of the floor construction to the top of the wall will depend upon the type and weight of the floor and the magnitude of the lateral load. For the usual foundation wall heights and depths below grade used in residences, special anchorage is seldom required with concrete slab, concrete joist or similar relatively heavy floor systems; the weight of the floor generally being considered as providing sufficient anchorage. With wood floors, however, the ends of the joists or beams should be securely anchored to the walls at intervals not exceeding 6 ft. in one and two story residences, and 4 ft. in other buildings. Similarly, floor joists or beams parallel and adjacent to foundation walls should be anchored to the walls with metal anchors at spacings not exceeding 8 ft. in one and two story residences, and 6 ft. in other buildings, with the anchors engaging at least three parallel joists. Suggested typical details for anchoring wood joists to concrete masonry basement walls are shown in Figs. 29 and 30.

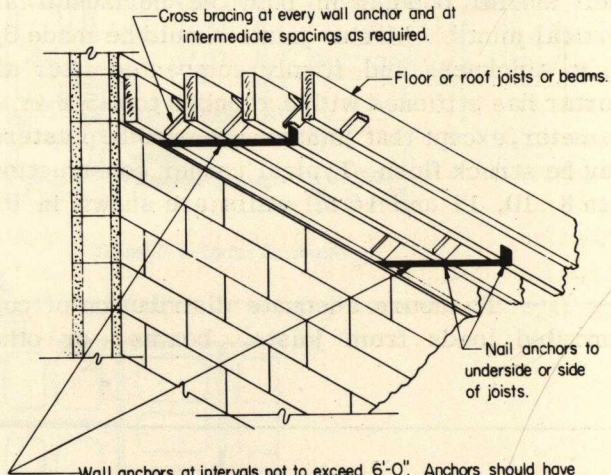

Cross bracing at every wall anchor and at intermediate spacings as required.
Floor or roof joists or beams.
Nail anchors to underside or side of joists.
Wall anchors at intervals not to exceed 6'-0". Anchors should have split end embedded in mortar joint or end bent down into block core and core filled with mortar. Length of anchor should be sufficient to engage at least three joists.

Figure 30 - Typical Detail for Anchorage of Concrete Masonry Walls to Parallel Wood Joists or Beams.

Crosswalls and partitions in basements should be anchored to the exterior basement walls at intersections if they are to serve as lateral supports for the exterior walls. In the past, it has been customary, and many codes still require, that intersecting bearing walls be laid up in a true masonry bond at intersections to insure that the walls will act together and mutually support one another in resisting lateral loads. It has been noted, however, that cracking sometimes develops in concrete masonry walls at or near masonry bonded intersections, apparently due to the reduced section at the "toothes" and the restraint to longitudinal wall

movements developed at such intersections. Accordingly, except in areas subject to severe earthquakes, it is recommended that concrete block walls not be tied together in a masonry bond except at corners. Instead, one wall should be terminated at the face of the other and the walls tied together by means of metal ties or bars, as illustrated in Fig. 31, of such size and at spacings as required to provide theoretically equivalent tensile and shearing resistance to that which could be obtained with a masonry bonded connection.

As a general rule, 1/4 in. thick, 1-1/4 in. wide and 30 in. long metal tie bars with 3 in. right angle bends should be used with bearing walls. The ties should be embedded in the horizontal mortar joints at vertical spacings not exceeding 32 in. with the bends at the end projecting into and embedded in cores filled with concrete or mortar. Metal lath or wire screen should be placed under the cores which will be filled to support the concrete mortar filling. Since the continuous vertical joint formed at the intersection may function as a control joint, the metal tie bars permitting some slight longitudinal movement of the walls, the mortar in the joint should be raked out to a depth of about 3/4 in. and filled with plastic caulking compound if the joint will be exposed to view. Essentially the same detail may be used to tie nonbearing partitions to exterior basement walls, except that if the partition is not intended to serve as a lateral support, strips of metal lath, galvanized hardware cloth or prefabricated wire ties

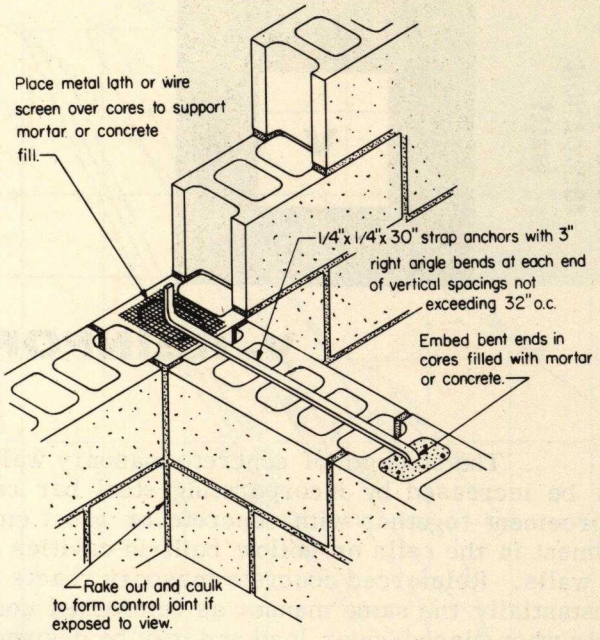

Place metal lath or wire screen over cores to support mortar or concrete fill.
1/4"x 1/4"x 30" strap anchors with 3" right angle bends at each end of vertical spacings not exceeding 32" o.c.
Embed bent ends in cores filled with mortar or concrete.
Rake out and caulk to form control joint if exposed to view.

Figure 31 - Suggested Method of Bonding Bearing Walls at Intersections.

in alternate courses may be substituted for the metal tie bars.

In localities where foundations must be designed to resist earthquake forces, the connections between intersecting walls should be given special attention. Bond beam or other horizontal reinforcing should be extended across the juncture a sufficient distance into the intersecting wall to develop proper anchorage. In addition, if the intersecting walls are not masonry bonded, substantial steel ties should be provided.

V. REINFORCED CONCRETE MASONRY FOUNDATION WALLS

The strength of concrete masonry walls can be increased by incorporating steel bar reinforcement together with concrete or grout embedment in the cells or hollow built-in cavities in the walls. Reinforced concrete masonry reacts in substantially the same manner as reinforced concrete when placed under load and may be designed in much the same way. The principal difference lies in the working stresses. The design tables and recommendations presented in the following are based on the applicable provisions and requirements of the Proposed American Standard Building Code Requirements for Reinforced Masonry, Fifth Draft, November 1954 Revision. Although this proposed code has not as yet (1957) been approved and accepted as an American Standard, it is one of the few authoritative sources presently available on the design of reinforced masonry.

Recommended applications for reinforced concrete masonry in foundation work lie principally in deep basement walls extending more than about 7 ft. below grade, foundation walls which are to support heavy uniform or concentrated loads, or foundation walls generally where the unsupported length or height will exceed that recommended for plain masonry walls of practical and economical thickness. It will be difficult to justify the use of structural reinforcing in concrete masonry foundation walls in most residential and light commercial construction except in those areas where seismic forces, extremely poor soil, or other unusual forces or conditions must be considered. Occasionally, however, the designer is confronted with the problem of designing foundation walls to support unusually heavy loads or to span unusually long distances between lateral supports. The information presented in the following is intended primarily to point out the manner in which concrete masonry can be applied to meet such special conditions.

A. DESIGN TABLES AND RECOMMENDATIONS

Tables 14 and 15, pages 38 to 41, present suggested designs for 8, 10, and 12 in. thick reinforced concrete masonry basement walls for various combinations of wall height and depth below grade. Suggested designs are given for two assumed soil conditions: Type A soil with an equivalent-fluid weight of 25 lb. per cu. ft.; and Type B soil with an equivalent-fluid weight of 40 lb. per cu. ft. In the preparation of these tables, it was assumed that the walls would act as simple beams supported at the top and bottom in resisting lateral earth pressures, and that materials and horizontal reinforcement as indicated above the tables would be used in the wall construction. It was further assumed that any vertical compressive loads carried by the walls could be considered as axially applied and uniformly distributed over the wall length, and would not exceed 5000 lb. per lin. ft. The recommended wall sections given in the tables have been so proportioned that for the assumed load and design conditions: 1) the calculated stresses do not exceed the allowable working stresses recommended in Table 16; and 2) the calculated deflections do not exceed 1/600 of the wall height between supports. Also, wall heights (H) and wall depths below grade (h) have been limited so that the sum of the two does not, in any case, exceed 18, 22, or 26 ft. with the 8, 10, and 12 in. thick walls respectively. As will be explained later, these values are reduced for the partially reinforced walls.

The vertical reinforcement recommended in concrete masonry basement walls for the same load and design conditions and the same range of wall height, depth below grade and thickness as assumed in Tables 14 and 15, may also be taken from the curves presented in Figs. 32 to 36. These curves may be used in the design of walls with heights and/or depths below grade different from those given in the tables. They may also be used in the selection of alternate bar sizes and spacings where the most economical section will be determined by the spacing of the bars rather than by the weight of reinforcement used in the section. When referring to these curves, it should be noted that the allowable depth below grade for any given combination of wall thickness, height, and vertical reinforcement is indicated at the bottom of the figures for Type A soil and at the

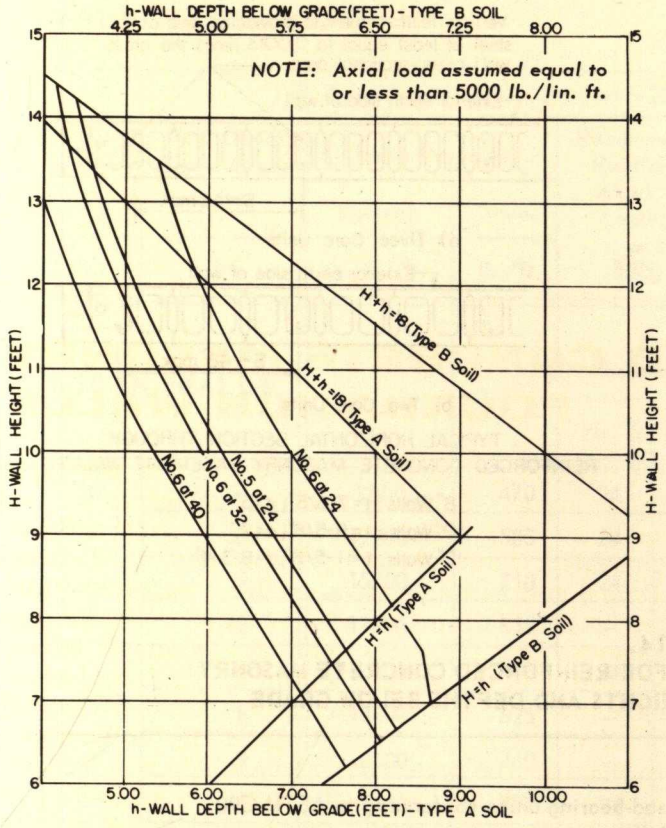

h-WALL DEPTH BELOW GRADE(FEET)-TYPE B SOIL
4.25 5.00 5.75 6.50 7.25 8.00
NOTE: Axial load assumed equal to or less than 5000 lb./lin. ft.
H+h=18(Type B Soil)
H+h=18(Type A Soil)
No.6 at 40
No.6 at 32
No.5 at 24
No.6 at 24
H=h (Type A Soil)
H=h (Type B Soil)
H-WALL HEIGHT (FEET)
5.00 6.00 7.00 8.00 9.00 10.00
h-WALL DEPTH BELOW GRADE(FEET)-TYPE A SOIL

Figure 32 - Recommended Vertical Reinforcement for 8 in. Reinforced Concrete Masonry Basement Walls.

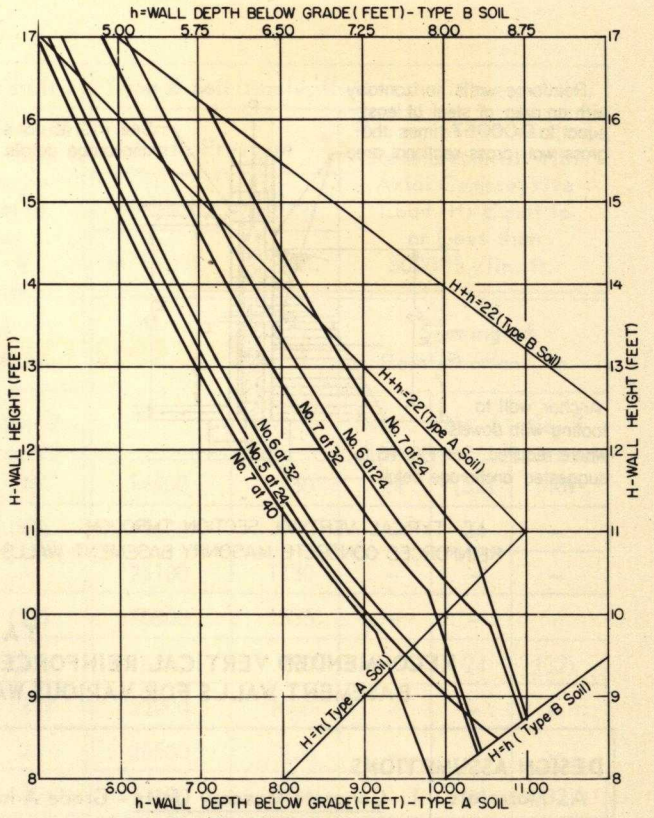


Figure 33 - Recommended Vertical Reinforcement for 10 in. Reinforced Concrete Masonry Basement Walls.

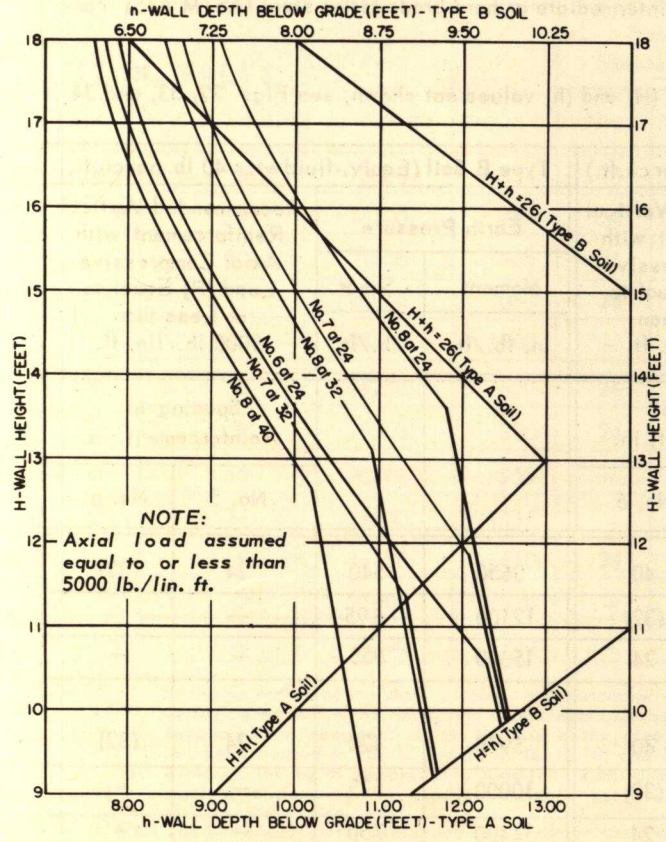
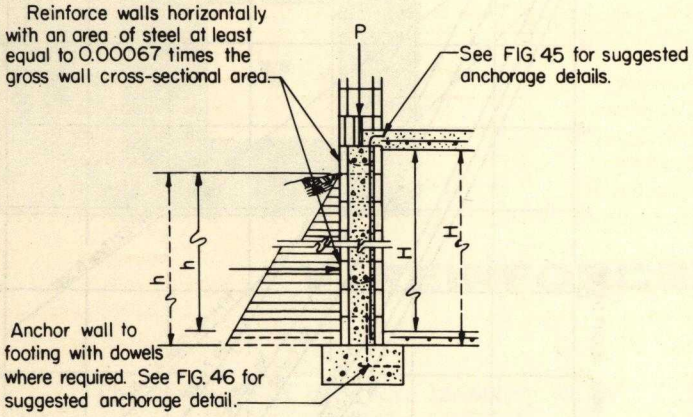

h-WALL DEPTH BELOW GRADE(FEET)-TYPE B SOIL
6.50 7.25 8.00 8.75 9.50 10.25
H+h=26(Type B Soil)
H+h=26(Type A Soil)
No.8 at 40
No.7 at 32
No.6 at 24
No.8 at 32
No.8 at 24
No.7 at 24
NOTE:
Axial load assumed equal to or less than 5000 lb./lin. ft.
H=h(Type A Soil)
H=h(Type B Soil)
H-WALL HEIGHT(FEET)
8.00 9.00 10.00 11.00 12.00 13.00
h-WALL DEPTH BELOW GRADE(FEET)-TYPE A SOIL

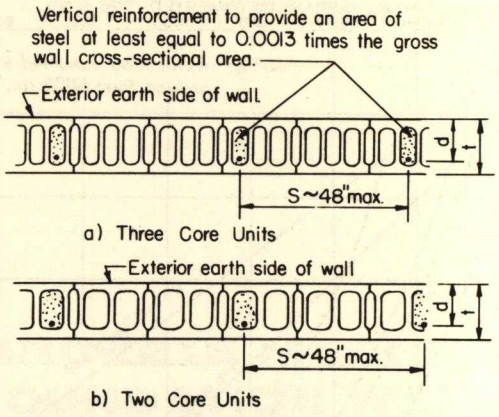
Figure 34 - Recommended Vertical Reinforcement for 12 in. Reinforced Concrete Masonry Basement Walls.

top for Type B soil designs. Also, when selecting the reinforcement recommended for a given combination of wall thickness, height and depth below grade, and soil type, any combination of bar size and spacing represented by curves to the right of the intersection of the wall height and depth below grade coordinates will be satisfactory, and combinations represented by curves to the left, unsatisfactory.

Although use of three-core units was assumed in the selection of the vertical reinforcement, the design tables and charts may also be used in the design of walls built of two-core units or units of any other design which will result in continuous clear core dimensions equal to or exceeding those obtained with three-core units. Also, the size and spacing of the reinforcement may be varied from that indicated, provided the area of steel furnished is at least equal to that recommended. For example, it may be desirable in some cases, to use two No. 5 bars in adjacent filled cores in lieu of one No. 7 bar in a single filled core at the indicated spacing to insure adequate clearance between the reinforcement and the masonry and to facilitate filling of the cores. In this regard, the recommended reinforcement generally will be conservative in any design in which the width of the solidly filled sections will exceed that assumed herein, based on the use of three-core units, or where more than one bar is used to provide the


Reinforce walls horizontally with an area of steel at least equal to 0.00067 times the gross wall cross-sectional area.
P
See FIG. 45 for suggested anchorage details.
h
h
H
H
Anchor wall to footing with dowels where required. See FIG. 46 for suggested anchorage detail.

TYPICAL VERTICAL SECTION THROUGH
REINFORCED CONCRETE MASONRY BASEMENT WALLS


Vertical reinforcement to provide an area of steel at least equal to 0.0013 times the gross wall cross-sectional area.
Exterior earth side of wall
d
S~48"max.
a) Three Core Units
Exterior earth side of wall
d
S~48"max.
b) Two Core Units

TYPICAL HORIZONTAL SECTION THROUGH
REINFORCED CONCRETE MASONRY BASEMENT WALLS

8" Walls: t = 7-5/8"; d=5"
10" Walls: t = 9-5/8"; d=7"
12" Walls: t = 11-5/8"; d=8-3/4"

TABLE 14.
RECOMMENDED VERTICAL REINFORCEMENT FOR REINFORCED CONCRETE MASONRY BASEMENT WALLS FOR VARIOUS WALL HEIGHTS AND DEPTHS BELOW GRADE

DESIGN ASSUMPTIONS

A. Materials - 1. Concrete Masonry Units - Grade A hollow load-bearing units conforming to ASTM C90.
2. Mortar - Type M(A-1) conforming to ASTM C270.
3. Core Fill - Grout or pea-gravel concrete with an ultimate compressive strength (28 days) of at least 2500 psi.
4. Reinforcement - Rail Steel (ASTM A16), or intermediate or hard grade billet steel (ASTM A15) conforming to ASTM A305.

B. Working Stresses - As shown in Table 16.

C. Design Calculations - See Appendix B.

For other (H) and (h) values not shown, see Figs. 32, 33, and 34.

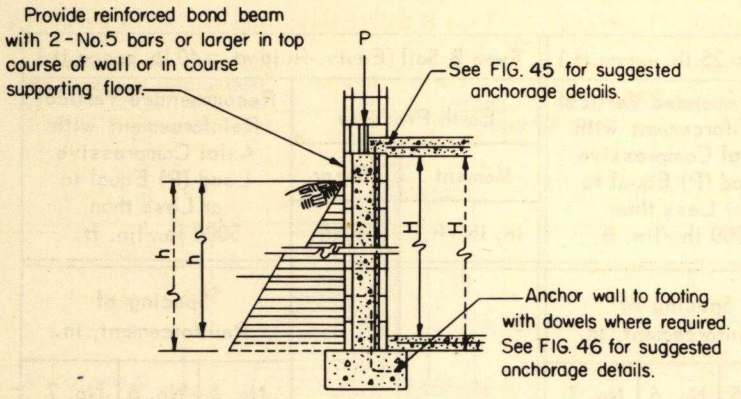
Wall Height H	Wall Depth Below Grade h	**Type A Soil** (Equiv.-fluid wt. = 25 lb. per cu. ft.)				**Type B Soil** (Equiv.-fluid wt. = 40 lb. per cu. ft.)			
		Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.		Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.	
		Moment	Shear			Moment	Shear		
ft.	ft.	in. lb./ft.	lb./ft.	Spacing of Reinforcement, in.		in. lb./ft.	lb./ft.	Spacing of Reinforcement, in.	
				No. 5	No. 6			No. 5	No. 6
8 IN. WALLS									
8	6	5400	340	–	40	8650	540	24	(32)
	7	7500	435	24	(32)	12100	695	–	–
	8	9800	535	–	24	15800	855	–	–
10	5	3690	260	–	40	5910	420	24	(32)
	6	6250	360	24	(32)	10000	575	–	–
	7	9000	470	–	24	14400	750	–	–

TABLE 14 (Continued)

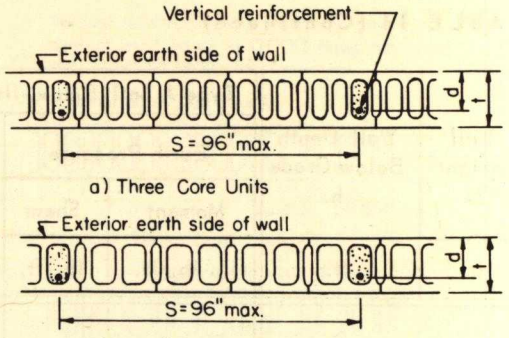
Wall Height H	Wall Depth Below Grade h	**Type A Soil** (Equiv.-fluid wt. = 25 lb. per cu.ft.) Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.			**Type B Soil** (Equiv.-fluid wt. = 40 lb. per cu.ft.) Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.		
		Moment	Shear				Moment	Shear			
ft.	ft.	in. lb./ft.	lb./ft.				in. lb./ft.	lb./ft.			
10 IN. WALLS				Spacing of Reinforcement, in.					Spacing of Reinforcement, in.		
				No. 5	No. 6	No. 7			No. 5	No. 6	No. 7
10	7	9000	470	24	(32)	(40)	14400	750	24	(32)	(40)
	8	12200	585	24	(32)	(40)	19600	940	–	24	–
	9	15700	710	24	(32)	–	25100	1130	–	–	–
	10	19200	835	–	24	(32)	30800	1335	–	–	–
12	7	10100	495	24	(32)	(40)	16400	790	–	24	(32)
	8	13800	625	–	24	(32)	22200	1000	–	–	–
	9	18100	760	–	–	24	29500	1220	–	–	–
14	6	7400	390	24	(32)	(40)	11820	620	–	24	(32)
	7	11000	510	–	24	(32)	17800	815	–	–	–
	8	15200	650	–	–	24	24500	1035	–	–	–
12 IN. WALLS				Spacing of Reinforcement, in.					Spacing of Reinforcement, in.		
				No. 6	No. 7	No. 8			No. 6	No. 7	No. 8
12	8	13800	625	24	(32)	(40)	22200	1000	24	(32)	(40)
	9	18100	760	24	(32)	(40)	29500	1220	–	24	–
	10	22800	905	24	(32)	(40)	36700	1445	–	–	–
	11	28000	1050	24	(32)	–	44800	1680	–	–	–
	12	33100	1200	–	24	–	53200	1920	–	–	–
14	8	15200	650	24	(32)	(40)	24500	1035	–	(24)	32
	9	20300	800	24	(32)	(40)	32300	1270	–	–	24
	10	26000	955	–	(24)	32	41700	1525	–	–	–
	11	31800	1115	–	–	24	51500	1785	–	–	–
16	8	16200	670	24	(32)	(40)	26100	1065	–	–	24
	9	21800	825	–	(24)	32	34800	1315	–	–	–
	10	28200	990	–	–	24	45200	1585	–	–	–

NOTES:

1. *In the selection of the wall sections recommended in the table above, use of materials, construction and working stresses as indicated above the table, and a maximum allowable deflection under sustained loading of 1/600 the wall height between supports were assumed. Also, the sum of the clear wall height between supports (H) plus the wall depth below grade (h) has been limited to 18, 22 and 26 ft. for 8, 10, and 12 in. walls respectively.*
2. *Although the axial compressive load (P) is assumed to be equal to or less than 5000 lb. per lin. ft. of wall, the required vertical reinforcement for smaller vertical loads will be the same since deflection and shear are the governing factors in the wall sections considered.*
3. *The bar spacings recommended in the above table represent the most economical spacing with respect to area or weight of steel required. Values shown in paranthesis represent alternate spacings (for the different bar sizes indicated) which may be used when it is desired to space reinforcement further apart.*
4. *Where it is desired to use vertical reinforcement at spacings exceeding 48 in. o.c., reference should be made to Table 15 or Figs. 35 and 36 and the recommended allowable wall heights and depths below grade given therein for partially reinforced concrete masonry basement walls.*


Provide reinforced bond beam with 2-No.5 bars or larger in top course of wall or in course supporting floor.
P
See FIG. 45 for suggested anchorage details.
h
h
H
H
Anchor wall to footing with dowels where required. See FIG. 46 for suggested anchorage details.

TYPICAL VERTICAL SECTION-PARTIALLY REINFORCED CONCRETE MASONRY BASEMENT WALLS



TYPICAL HORIZONTAL SECTION-PARTIALLY REINFORCED CONCRETE MASONRY BASEMENT WALLS

10" Walls: t = 9-5/8"; d = 7"
12" Walls: t = 11-5/8"; d = 8-3/4"

TABLE 15.
RECOMMENDED VERTICAL REINFORCEMENT FOR PARTIALLY REINFORCED CONCRETE MASONRY BASEMENT WALLS FOR VARIOUS WALL HEIGHTS AND DEPTHS BELOW GRADE

DESIGN ASSUMPTIONS

A. Materials - 1. Concrete Masonry Units - Grade A hollow load-bearing units conforming to ASTM C90.
2. Mortar - Type M (A-1) conforming to ASTM C270.
3. Core Fill - Grout or pea-gravel concrete with an ultimate compressive strength (28 days) of at least 2500 psi.
4. Reinforcement - Rail steel (ASTM A16), or intermediate or hard grade billet steel (ASTM A15) conforming to ASTM A305.

B. Working Stresses - As shown in Table 16 except as noted below.

C. Design Calculations - See Appendix B. For other (H) and (h) values not shown, see Figs. 35 and 36.

Wall Height H	Wall Depth Below Grade h	**Type A Soil** (Equiv.-fluid wt. = 25 lb. per cu.ft.) Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.			**Type B Soil** (Equiv.-fluid wt. = 40 lb. per cu.ft.) Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.		
		Moment	Shear				Moment	Shear			
ft.	ft.	in. lb./ft.	lb./ft.	Spacing of Reinforcement, in.			in. lb./ft.	lb./ft.	Spacing of Reinforcement, in.		
10 IN. WALLS				No. 5	No. 6	No. 7			No. 5	No. 6	No. 7
8	7	7500	435	56	(64)	(72)	12100	695	32	(40)	(48)
	8	9800	535	40	(48)	(56)	15800	855	–	–	–
10	6	6250	360	56	(72)	(88)	10000	575	(32)	48	(56)
	7	9000	470	40	(56)	(64)	14400	750	–	–	–
	8	12200	585	32	(40)	(48)	19600	940	–	–	–
12	5	4150	270	56	(72)	(88)	6650	435	40	(56)	(64)
	6	6900	375	40	(56)	(64)	11000	600	–	–	–
	7	10100	495	32	(40)	(48)	16400	790	–	–	–

TABLE 15 (Continued)

Wall Height H	Wall Depth Below Grade h	Type A Soil (Equiv.-fluid wt. = 25 lb. per cu. ft.)						Type B Soil (Equiv.-fluid wt. = 40 lb. per cu. ft.)					
		Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.				Earth Pressure		Recommended Vertical Reinforcement with Axial Compressive Load (P) Equal to or Less than 5000 lb./lin. ft.			
		Moment	Shear					Moment	Shear				
ft.	ft.	in. lb./ft.	lb./ft.					in. lb./ft.	lb./ft.				
12 IN. WALLS				Spacing of Reinforcement, in.						Spacing of Reinforcement, in.			
				No.5	No.6	No.7	No.8			No.5	No.6	No.7	No.8
10	7	9000	470	56	(72)	(88)	–	14400	750	40	(48)	(56)	(64)
	8	12200	585	48	(64)	(72)	(80)	19600	940	32	(40)	–	–
	9	15700	710	40	(48)	(56)	(64)	25100	1130	24	(32)	–	–
	10	19200	835	32	(40)	(48)	(56)	30800	1335	–	–	–	–
12	7	10100	495	48	(64)	(80)	(88)	16400	790	32	(40)	(48)	(56)
	8	13800	625	(32)	48	(56)	(64)	22200	1000	24	(32)	(40)	–
	9	18100	760	24	(32)	(40)	–	29500	1220	–	–	–	–
	10	22800	905	24	(32)	(40)	–	36700	1445	–	–	–	–
14	7	11000	510	(32)	48	(56)	(64)	17800	815	24	(32)	(40)	–
	8	15200	650	24	(32)	(40)	–	24500	1035	–	–	–	–

NOTES: 1. *Use of materials, construction, and working stresses as indicated above the table and a maximum allowable deflection under sustained loading of 1/600 the wall height between supports were assumed with the following exception:*

a) *The allowable stress in axial compression was taken as 85 psi on the gross wall area as recommended in the design of non-reinforced concrete masonry walls.*

Also, the sum of the clear wall height between supports (H) plus the wall depth below grade (h) has been limited to 20 and 24 ft. for 10 and 12 in. walls respectively.

2. *Although the axial compressive load (P) is assumed to be equal to or less than 5000 lb. per lin. ft. of wall, the required vertical reinforcement for smaller vertical loads, in general, will be similar since deflection and shear (or bond) stresses are the governing factors in most of the wall sections considered above. For axial compressive loads exceeding 5000 lb. per lin. ft. of wall, the designer should check the steel and masonry stresses and wall deflection.*

3. *The bar spacings recommended in the above table represent the most economical spacing with respect to area or weight of steel required. Values shown in paranthesis represent alternate spacings (for the different bar sizes indicated) which may be used when it is desired to space reinforcement further apart.*

4. *For recommended reinforcement requirements for reinforced concrete masonry basement walls, reference should be made to Table 14 or Figs. 32, 33, and 34.*

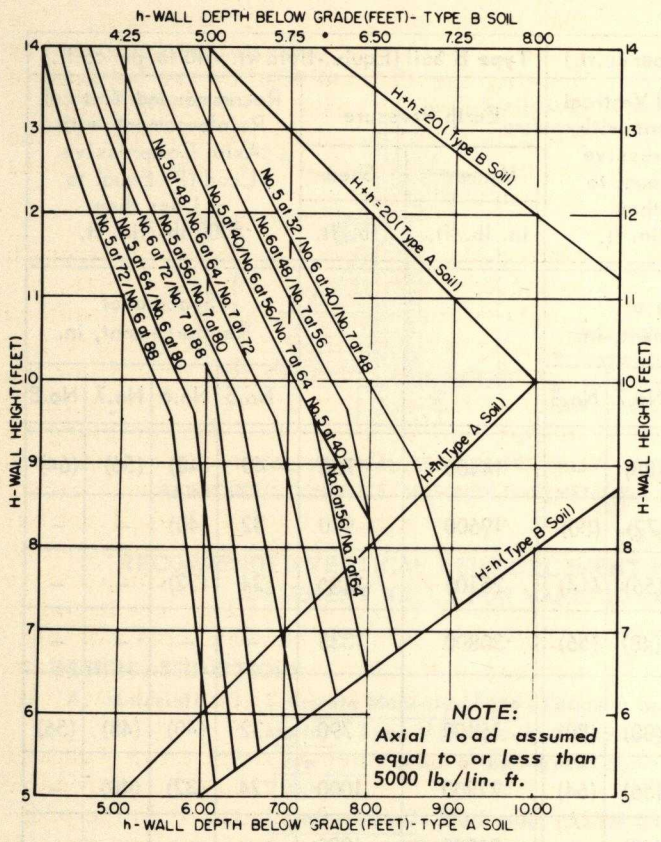

h-WALL DEPTH BELOW GRADE(FEET)- TYPE B SOIL
4.25
5.00
5.75
6.50
7.25
8.00
H+h=20 (Type B Soil)
H+h=20 (Type A Soil)
No 5 at 32/No 6 at 40/No 7 at 48
No 6 at 48/No 7 at 56
No 5 at 40/No 6 at 56/No 7 at 64
No 5 at 48/No 6 at 64/No 7 at 72
No 5 at 56/No 7 at 80
No 6 at 72/No 7 at 88
No 5 at 64/No 6 at 80
No 5 at 72/No 6 at 88
H=h (Type A Soil)
H=h (Type B Soil)
NOTE:
Axial load assumed equal to or less than 5000 lb./lin. ft.
H-WALL HEIGHT (FEET)
5.00
6.00
7.00
8.00
9.00
10.00
h-WALL DEPTH BELOW GRADE(FEET)- TYPE A SOIL

Figure 35 - Recommended Vertical Reinforcement for 10 in. Partially Reinforced Concrete Masonry Basement Walls.

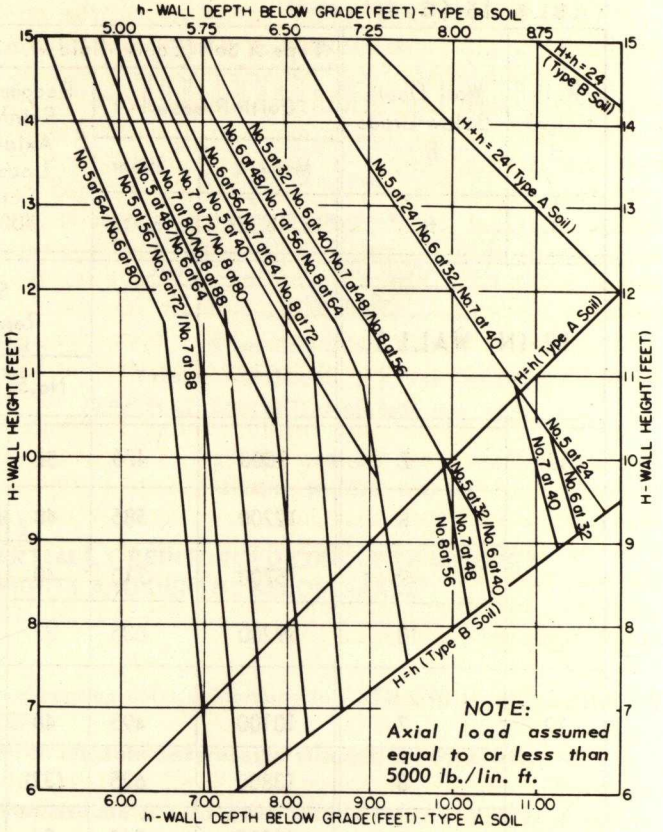

h-WALL DEPTH BELOW GRADE(FEET)-TYPE B SOIL
5.00
5.75
6.50
7.25
8.00
8.75
H+h=24 (Type B Soil)
H+h=24 (Type A Soil)
No 5 at 64/No 6 at 80
No 5 at 56/No 6 at 72/No 7 at 88
No 5 at 48/No 6 at 64/No 7 at 80/No 8 at 88
No 7 at 72/No 8 at 80
No 5 at 40/No 6 at 48/No 7 at 56/No 8 at 64
No 5 at 32/No 6 at 40/No 7 at 48/No 8 at 56
No 5 at 24/No 6 at 32/No 7 at 40
H=h (Type A Soil)
No 5 at 24
No 6 at 32
No 7 at 40
No 5 at 32/No 6 at 40
No 7 at 48
No 8 at 56
H=h (Type B Soil)
NOTE:
Axial load assumed equal to or less than 5000 lb./lin. ft.
H-WALL HEIGHT (FEET)
6.00
7.00
8.00
9.00
10.00
11.00
h-WALL DEPTH BELOW GRADE(FEET)-TYPE A SOIL

Figure 36 - Recommended Vertical Reinforcement for 12 in. Partially Reinforced Concrete Masonry Basement Walls.

recommended area of steel reinforcement in the section.

Typical examples and calculations, illustrating the use of the tables and curves and the design procedures, are given in Appendix B. In the event that actual design or load conditions differ appreciably from those assumed, a special design will be required. This is particularly true with respect to walls which are to support axial compressive loads in excess of 5000 lb. per lin. ft., eccentric compressive loads producing substantial moments which act with rather than against the earth pressure moment, or loads due to earthquake or blast forces. Also, where basement walls are laterally supported by vertical elements at horizontal intervals less than about two times the wall height, consideration should be given to the extent to which these supports will promote horizontal distribution of the lateral earth pressure load and act to reduce the vertical moment.

1. Minimum Reinforcement Requirements

The minimum amount of reinforcement generally recommended in a reinforced masonry wall is 0.002 times the gross cross-sectional area, with not more than 2/3 of this minimum area placed in one direction. For example, the minimum area of steel recommended in a wall 8 in. in nominal thickness is 0.183 sq. in. per lin. ft. (0.002 x 7.625 x 12). If the wall is to act as a one-way beam or slab in resisting bending, not more than 2/3 of this minimum area should be placed in the direction of principal bending, or conversely, not less than 1/3 of the minimum area should be placed transverse to the direction of principal bending. If, in the preceding example, the principal bending will be in the vertical span, not more than 0.122 sq. in. per ft. of the minimum area of reinforcement required may be used in the vertical direction, and not less than 0.061 sq. in. per ft. in the horizontal direction.

The maximum spacing of the vertical reinforcement in reinforced concrete masonry walls should not exceed 48 in. Horizontal reinforcement may be concentrated in bond beams spaced 4 to 6 ft. apart, or located at strategic points in the wall height, as above and below openings, at roof and floor levels and at the top of parapet walls. Only horizontal reinforcement which is continuous in the wall should be considered in computing the minimum area of reinforcement. In walls laid in stacked and similar bond patterns, the spacing of the vertical reinforcement should not exceed the length between continuous vertical joints so that each stack or column of units will be reinforced.

Where the area or the spacing of the reinforcement does not conform to that recommended, the walls should be considered as partially reinforced. Partially reinforced walls are designed in the same manner as plain masonry walls, except that the reinforcement is considered effective in resisting flexural stresses. In partially reinforced walls, vertical reinforcement should be provided on each side of any openings and at each wall corner, and at horizontal intervals not exceeding 8 ft. between such points. The walls should also be reinforced horizontally above and below openings, at floor and roof levels, and at the top of parapet walls.

The horizontal reinforcement suggested in the details above Tables 14 and 15 constitutes the minimum recommended in walls of these types. Where it is expected that high horizontal stresses will be developed in such walls due to volume change, restraint at vertical supports or at building corners, or other reasons, additional horizontal reinforcement may be required.

2. Allowable Stresses

Recommended allowable stresses for reinforced concrete masonry are given in Table 16 on the next page. As indicated in this table, allowable axial and flexural compressive stresses are based on the ultimate strength of the unreinforced masonry (f'_m). This strength may be determined by actual test of small masonry piers, or based on the ultimate compressive strength of the masonry units on the net section and the type of mortar used to lay up the units. The recommended alternate methods of determining (f'_m) in the design of reinforced concrete masonry are:

Method No. 1: When the strength is to be determined by test of masonry assemblages, the tests shall be made well in advance of construction using prisms built of similar materials, under the same conditions, and insofar as possible, with the same bonding arrangements as will be used in the structure.

The test prisms shall be built in the form of hollow squares, 8 by 8 in. in plan and 16 in. high, or in the form of rectangles, 8 by 16 in. in plan and 16 in. high. The hollow cores shall not be filled with concrete or grout.

After construction, the test prisms shall be damp stored for three days and then stored in air at a temperature of not less than 65 F. The prisms shall then be capped and tested in accordance with the standard methods prescribed in Standard Method for Determining Compressive Strength of Molded Concrete Cylinders, ASTM C39. The standard age of specimens at time of test shall be 28 days with not less than three specimens for each test. The masonry strength (f'_m) shall be computed by dividing the ultimate load by the net area of the masonry units used in the construction of the prism and taken as the average of the results obtained with at least three specimens in each test group.

Method No. 2: When the masonry strength is not determined by preliminary tests, allowable stresses may be based on an assumed masonry strength (f'_m) not exceeding 60 per cent of the net compressive strength of the units on the net section where Type M (A-1) mortar will be used, and not exceeding 45 per cent of the compressive strength when Type S (A-2) mortar will be used, except that such assumed value (f'_m) shall not exceed 2,000 psi or 1,500 psi respectively. Tests to determine the net compressive strength of the units shall be conducted in accordance with the standard methods prescribed in Standard Methods of Sampling and Testing Concrete Masonry Units, ASTM C140.

Allowable stresses as given in Table 16 for an assumed masonry ultimate compressive strength (f'_m) of 1600 psi were used in the preparation of the recommended wall designs given in Table 14 and the curves in Figs. 32, 33 and 34. These same stresses were used in calculating the design flexural resistance of the partially reinforced wall sections shown in Table 15 and in Figs. 35 and 36, except that the allowable stress in axial compression was taken as 85 psi on the gross wall section as recommended in the design of unreinforced walls.

The value of 1600 psi assumed for the ultimate compressive strength of the masonry in this booklet is the value permitted by the proposed American Standard code with units testing 2667 psi or higher on the net area and laid up with a Type M (A-1) mortar, when the strength of the masonry is not determined by actual test. Some test data are available and are presented in Table 17, page 44, which indicate that the assumption of an ultimate compressive strength equal to 60 per cent of the unit strength on the net area is conservative. Principal details of the test piers are shown in Fig. 37, page 45. These data suggest that the assumption of an ultimate masonry compressive strength of 80 per cent of the unit strength may not be unreasonable if the ultimate strength of two block high piers can be taken as representative of the ultimate masonry strength. On this basis, the assumed value of 1600 psi is representative of construction with units testing 2000 psi on the net area, or about the strength of hollow load-bearing concrete masonry units meeting the usual ultimate compressive strength requirement of 1000 psi on the gross area for Grade A units.

TABLE 16.
RECOMMENDED ALLOWABLE STRESSES IN REINFORCED CONCRETE MASONRY BASED ON THE PROPOSED AMERICAN STANDARD BUILDING CODE REQUIREMENTS FOR REINFORCED MASONRY, FIFTH DRAFT; REVISED, NOV. 1954

Type of Stress		Working Stresses *	
		Masonry Strength -- f'_m	f'_m = 1,600 psi
Compression -- Axial	f_m =	0.20 f'_m †	320 psi †
Compression - Flexural	f_m =	0.33 f'_m	528 psi
Shear--No Shear Reinforcement	v_m =	0.02 f'_m ‡ (40 psi max)	32 psi
--With Properly Designed Shear Reinforcement	v =	0.04 f'_m ‡ (120 psi max)	64 psi
Bearing	f_m =	0.25 f'_m	400 psi
Bond--Plain Bars			
Type M (A-1) Mortar and Grout	u =	80 psi	80 psi
Type S (A-2) Mortar and Grout	u =	60 psi	60 psi
Deformed Bars			
Type M (A-1) Mortar and Grout	u =	160 psi	160 psi
Type S (A-2) Mortar and Grout	u =	120 psi	120 psi
Modulus of Elasticity	E_m =	1,000 f'_m	1,600,000 psi
Tensile Stress in Longitudinal and Web Reinforcement-- Intermediate and Hard Grade Bars	f_s =	20,000 psi	20,000 psi

Combined Stresses: $\frac{f_m}{F_m} + \frac{f_a}{F_a}$ not to exceed 1.0

Where: f_a = Computed axial unit stress; F_a = Axial unit stress permitted by Code.

f_m = Computed flexural unit stress; F_m = Flexural unit stress permitted by Code.

* Working stresses may be increased by 33-1/3 per cent where designing for wind load in combination with dead and other loads providing the calculated stresses due to dead and other loads alone do not exceed the allowable stresses.

† In reinforced bearing walls reinforced with an area of steel not less than 0.002 times the gross cross-sectional area of the wall, not more than 2/3 of which is used in the principal direction and where the principal reinforcement is spaced not more than 48 in. apart. Where the reinforcement is designed, placed and anchored in position as for tied columns, higher stresses are permitted.

‡ Allowable shear stress coefficients based on Uniform Building Code.

TABLE 17.
ULTIMATE COMPRESSIVE STRENGTH OF TWO AND FIVE BLOCK HIGH HOLLOW CONCRETE MASONRY UNIT PIERS *

Pier Height (No. of Block)	Block Units		Mortar Type	Ultimate Compressive Strength of Piers, † f'_p, psi.	Ratio Pier Strength to Unit Strength, f'_p/f'_b
	Size	Aggregate Type			
	Half Block				
2	8 x 8 x 8 in.	Sand & Gravel	M(A-1)	3710	0.86
2	8 x 8 x 8 in.	Sand & Gravel	N (B)	3490	0.81
5	8 x 8 x 8 in.	Sand & Gravel	M(A-1)	3630	0.84
5	8 x 8 x 8 in.	Sand & Gravel	N (B)	3360	0.78
2	8 x 8 x 8 in.	Haydite	M(A-1)	2640	0.87
5	8 x 8 x 8 in.	Haydite	M(A-1)	2600	0.86
	Column				
2	12 x 8 x 12 in.	Sand & Gravel	M(A-1)	1980	0.87
2	12 x 8 x 12 in.	Haydite	M(A-1)	2300	0.91

* Date from tests conducted at the Univ. of Wisconsin and reported in "Investigation of the Structural Properties of Reinforced Concrete Masonry", published by NCMA.

† Based on the average net cross-sectional area of the block units with the cores empty.

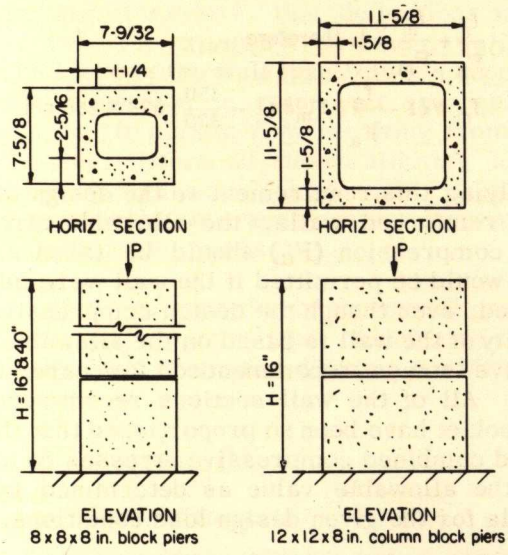


Figure 37 - Principal Details of Piers Tested for Ratio, f'_p/f'_b and Ratio, E_p/E_b.

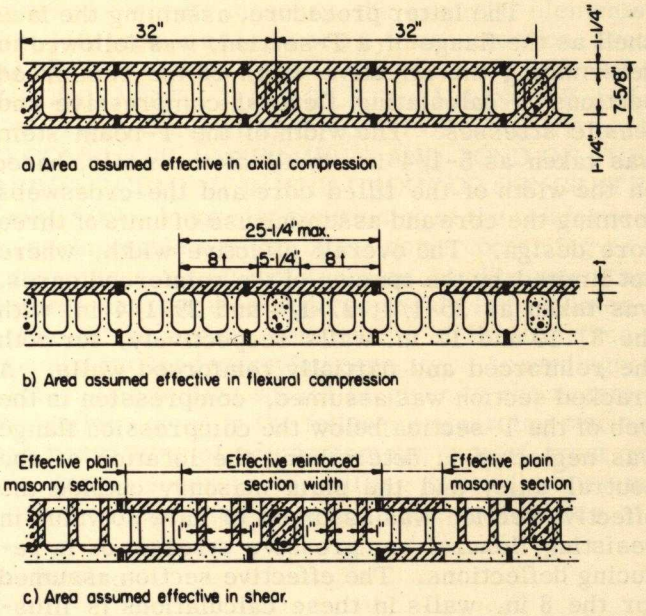


Figure 38 - Effective Sections Assumed for Stress Calculations in the 8 in. Reinforced Concrete Masonry Basement Wall Designs.

3. Minimum Wall Thickness and Lateral Support

Reinforced concrete masonry foundation walls should be of a thickness as required to support the expected vertical and lateral loads without exceeding the allowable stresses. If not subject to lateral earth pressure or other sustained lateral loads, the walls should be not less than 6 in. in nominal thickness. Basement walls should be not less than 8 in. in nominal thickness.

In the design of reinforced concrete masonry basement walls, it is recommended that the clear wall height (H), between the supports at the top and bottom, and the wall depth below grade (h) be limited so that the sum of the two does not exceed 18, 22, or 26 ft. with 8, 10 and 12 in. thick walls respectively, or 25 times the nominal wall thickness (25t) with walls over 12 in. thick. For partially reinforced basement walls, H+h should not exceed 20 and 24 ft. respectively for the 10 in. and 12 in. thick walls.

4. Effective Sections for Stress Calculations

The working stresses recommended in the design of reinforced concrete masonry walls apply to the net section of the walls considered effective in resisting stress. With hollow unit wall construction, the effective net section will vary and generally will be dependent upon the thickness of the face shells and cross webs of the units, and on the type of mortar bedding employed in the construction. If the walls are laid up with face shell mortar bedding with only those crosswebs adjacent to and forming the filled cores bedded in mortar, the net section effective in resisting axial compressive loads will be the hatched portion of the wall section shown in Fig. 38(a). As a general rule, minimum face shell thicknesses of 1-1/4, 1-3/8 and 1-1/2 in. and minimum crossweb thicknesses of 1, 1-1/8 and 1-1/4 in. with 8, 10 and 12 in. units, respectively, may be assumed for Grade A hollow load-bearing concrete masonry units, and these values were used in the calculations for this booklet.

Regarding the manner in which reinforced concrete masonry walls react in resisting bending, sufficient data are available to indicate that it can safely be assumed that some portion of the bending stresses will be distributed to and carried by the unreinforced portions of the wall between the reinforced sections. Some authorities recommend that the width of the section assumed effective in resisting flexural stress be arbitrarily limited so as not to exceed four times the wall thickness with solid unit construction nor one and one-half times the length of the individual wall units with hollow unit construction. A somewhat more logical method of establishing the effective width of the reinforced section for flexural stress calculations is based on the assumption that the reinforced sections may be considered as comparable to T-beams, and the effective width based on the same limitations as recommended in the design of reinforced concrete T-beams. The American Concrete Institute, for example, in "Building Code Requirements for Reinforced Concrete (ACI 318-56)," recommends that the effective width assumed in the design of T-beams be limited so that the overall flange does not exceed one-fourth the beam span and the flange overhang on either side of the stem does not exceed eight times the thickness of the flange nor one-half the clear distance between adjacent beams.

The latter procedure, assuming the face shell as the flange of a T-section, was followed in determining the effective width of the reinforced sections in calculating flexural compressive and tensile stresses. The width of the T-beam stem was taken as 5-1/4 in. in all of the walls, based on the width of the filled core and the crosswebs forming the core and assuming use of units of three core design. The overall effective width, where not limited by the spacing of the reinforced cores, was taken as 25-1/4, 27-1/4 and 29-1/4 in. with the 8, 10 and 12 in. walls respectively, for both the reinforced and partially reinforced walls. A cracked section was assumed, compression in the web of the T-section below the compression flange was neglected in determining the location of the neutral axis, and the plain masonry outside the effective section was assumed to have no value in resisting flexural compressive stresses or in reducing deflections. The effective section assumed for the 8 in. walls in these calculations is illustrated by the shaded portions of the section shown in Fig. 38(b).

Ordinarily, the overhanging portions of the flange of a T-beam are not considered effective in resisting diagonal tension and shear stresses and it is usually assumed that all of the shear will be taken by the web of the beam. Since the crosswebs of the units in hollow unit walls act to stiffen and add appreciably to the shearing resistance of the walls, however, an allowance should be made for this effect. Accordingly, the effective width used in calculating shear stresses was taken as the width of the reinforced core plus the width of any crosswebs falling within the effective section. Also, the shearing resistance of the plain masonry outside the effective section, based on the net area bedded in mortar and an allowable working stress in shear of 12 psi as recommended for the design of plain masonry, was included in determining the overall design shearing resistance of both the reinforced and partially reinforced walls. The effective section assumed for the 8 in. walls is illustrated in Fig. 38(c).

5. Combined Bending and Direct Stress

In the design of walls subject to both direct and flexural compressive stress, it is recommended that the walls be so proportioned that the summation of the ratios of the calculated to the allowable axial and flexural compressive stresses does not exceed unity. For example, given allowable stresses of 320 psi in axial compression (F_a) and 528 psi in flexural compression (F_m), and a calculated direct compressive stress (f_a) of 150 psi, the allowable flexural compressive stress (f_m) would be calculated as follows:

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} \leq 1, \text{ therefore}$$

$$f_m = (1 - \frac{f_a}{F_a}) F_m = (1 - \frac{150}{320}) 528 = 280 \text{ psi}$$

In applying this requirement to the design of partially reinforced walls, the allowable stress in axial compression (F_a) should be taken as that which would be permitted if the wall were fully reinforced, even though the design compressive load capacity of the wall is based on the allowable compressive stress recommended for unreinforced walls. All of the wall sections recommended in this booklet have been so proportioned that the calculated combined compressive stresses do not exceed the allowable value as determined by this formula for the given design load conditions.

When calculating flexural stresses produced in walls by lateral forces other than those due to earthquake movements, it will generally be permissible to assume that dead load compressive stresses will be effective in opposing and reducing the magnitude of vertical flexural tensile stresses. In designing for earthquake forces, however, not more than 50 per cent of the vertical dead load compressive stress should be considered as effective in opposing development of vertical tensile stresses.

The effect of axial compressive loads in increasing bending moments and deflections in laterally loaded walls should be given consideration in the design. Accordingly, the design bending moment was taken as equal to the earth pressure moment plus the moment which would be produced by the assumed axial load acting at an eccentricity equal to the maximum allowable deflection of 1/600 times the wall height.

6. Deflections

Deflections in reinforced concrete masonry basement walls should be limited to a reasonable value to prevent development of or to limit the width of cracks in the tension face of such walls. In this regard, some fine cracking may develop at the horizontal mortar joints in walls subject to vertical bending, even at relatively small deflections. These cracks will seldom be objectionable as long as they remain fine. Excessive opening of such cracks will be objectionable, however, since it may lead to penetration of moisture through the walls, disruption of finishes applied to the wall surface, or at the very least, detract from the appearance or cause undue concern regarding the structural stability of the wall construction. It is

recommended, therefore, that deflections in reinforced concrete masonry basement walls be limited to 1/600 of the wall span between supports. If proper allowance is made for the inelastic deflection due to plastic flow resulting from sustained loading, the use of such a limiting deflection in the design calculations will, ordinarily, provide reasonable assurance against development of excessively wide cracks.

In the preparation of these wall designs, deflections were computed from an equation developed by the late Professor George A. Maney in which:

$$D = \frac{CL^2}{d}(e_s + e_m)$$

where D = calculated deflection, in.

C = coefficient varying with the type of support and the type of loading

L = clear span, in.

d = effective depth, in.

$e_s = \frac{f_s}{E_s}$ = maximum tensile strain in the tensile reforcement

$e_m = \frac{f_m}{E_m}$ = maximum compressive unit strain in the extreme fiber of the masonry.

In the application of this formula, the allowable deflection was limited to L/600 using a coefficient (C) of 0.11 as recommended for simply supported beams under uniform load, and a sustained modulus of elasticity for the masonry (E'_m) of one-half the assumed instantaneous value (E_m).

B. RECOMMENDED CONSTRUCTION DETAILS AND PRACTICES

Typical details showing two methods of laying up reinforced concrete masonry walls are given in Fig. 39 and Fig. 40 on the following pages. Fig. 39 illustrates a method in which the vertical steel is set and the cores filled as the wall construction progresses, employing units with one open-end to permit setting of the units around the vertical steel. Fig. 40 shows a method in which the vertical steel is set and the cores filled after the wall has been built up. Either method may be used in most cases, the choice being dependent upon the availability of the various types of units and the contractor's preference. Generally, the method shown in Fig. 39 should be slightly more economical since cleanout openings at the base of the reinforced cells are not required and placement of steel and corefill may be accomplished more easily and expeditiously.

Regardless of the type of units, reinforcing or methods employed in the construction of reinforced concrete masonry, the essential elements for safe and satisfactory construction are: (1) PROPER POSITIONING OF THE STEEL REINFORCEMENT; 2) COMPLETE FILLING OF ALL CAVITIES CONTAINING REINFORCEMENT; and 3) COMPLETE EMBEDMENT AND BOND OF THE REINFORCEMENT IN THE GROUT OR CONCRETE COREFILL. Incorrect positioning of the reinforcement, incomplete filling of the cores containing reinforcement, or lack of adequate bond between the corefill and the steel will seriously impair interaction of the various components of the construction and its load-carrying capacity. Accordingly, particular attention should be given to the recommendations given in the following regarding setting and positioning of reinforcement, use of grout or pea-gravel concrete for filling cores, minimum clearances and maximum pour lifts and heights.

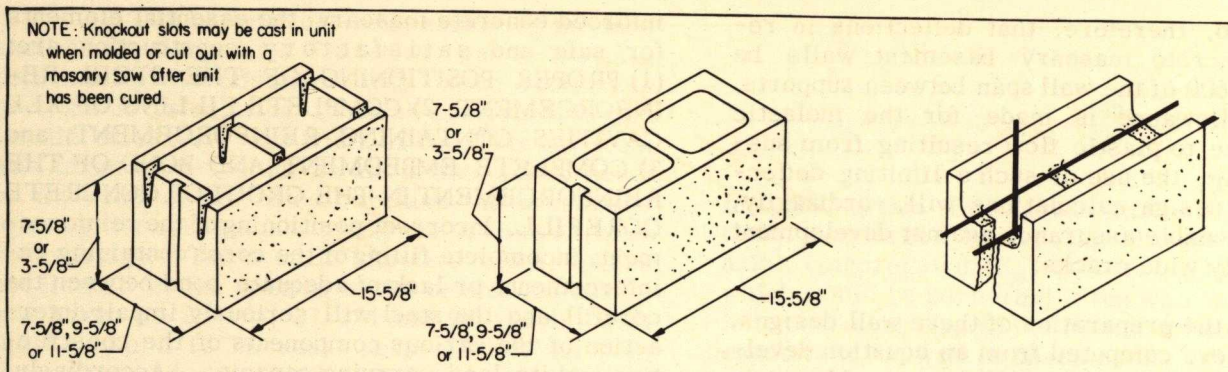
1. Footings

Recommendations on the design and construction of footings for plain concrete masonry foundation walls, given in a previous section of this booklet, also apply to footings for reinforced construction. Since the loads transmitted to footings supporting reinforced walls will usually be greater, larger footings will usually be required and the need for a reinforced footing often more easily justified. Also, in those cases where the footing alone serves to restrain the base of a wall subject to lateral pressure against horizontal movement, dowels between the footing and wall will be needed and the effect of any shear or moment on the soil pressure under and adjacent to the footing should be investigated.

2. Wall Construction

a. Units--Either two-core or three-core units may be used in the construction of reinforced concrete masonry, however, the vertical cells in walls built with two-core units will usually be larger and easier to reinforce and fill than those in walls built with three-core units. Typical dimensions of standard three-core and two-core concave-end units and typical clear dimensions of vertical cells in walls built with such units are shown in Fig. 41, page 50. Similar dimensions for plain-end or open-end two-core units are shown in Fig. 42, page 51. While two-core units, particularly of the plain-end or open-end types, offer some advantage in providing larger, more easily filled cores, they are not available in all areas or from all block producers in any one area. Consequently, designers should check with producers to insure that units of the type assumed and called for in the design will be available in the area in which the construction is to be located.

b. Laying Up the Walls--Whether two or three-core units are used, a full mortar bed

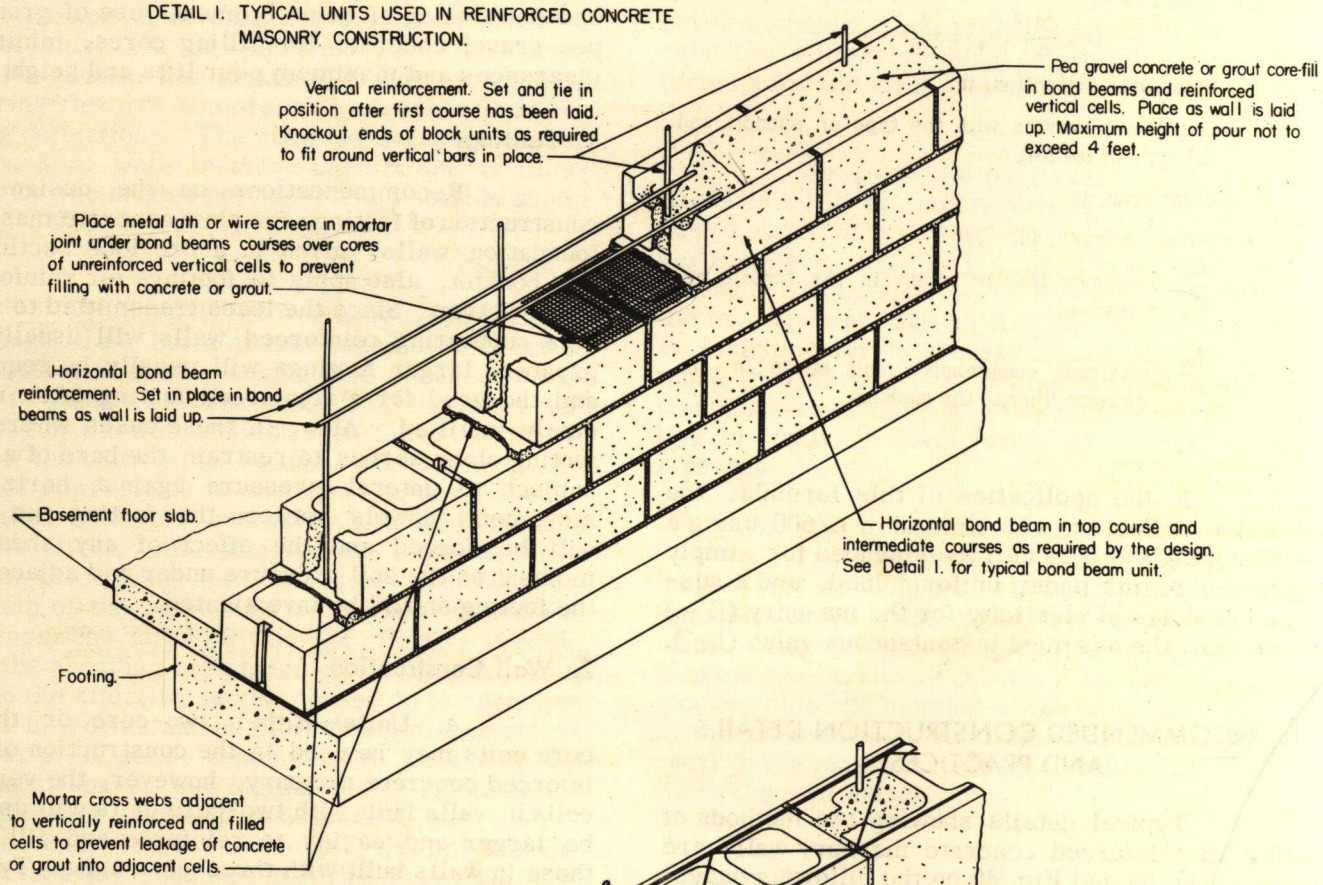

NOTE: Knockout slots may be cast in unit when molded or cut out with a masonry saw after unit has been cured.
7-5/8" or 3-5/8"
15-5/8"
7-5/8", 9-5/8" or 11-5/8"
7-5/8" or 3-5/8"
15-5/8"
7-5/8", 9-5/8" or 11-5/8"

a) Standard unit with end and web knockout slots.

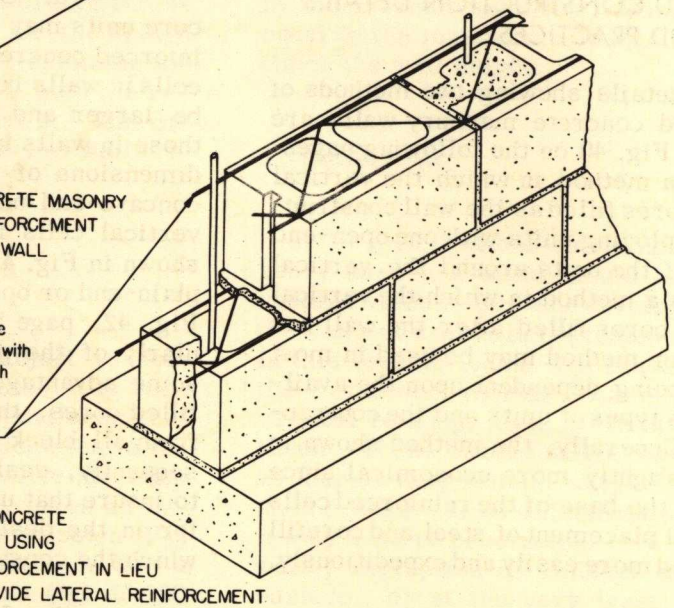
b) Standard unit with knockout slot in one end.

c) Standard unit with sections of end and cross webs removed to permit placement of reinforcing.

DETAIL 1. TYPICAL UNITS USED IN REINFORCED CONCRETE MASONRY CONSTRUCTION.

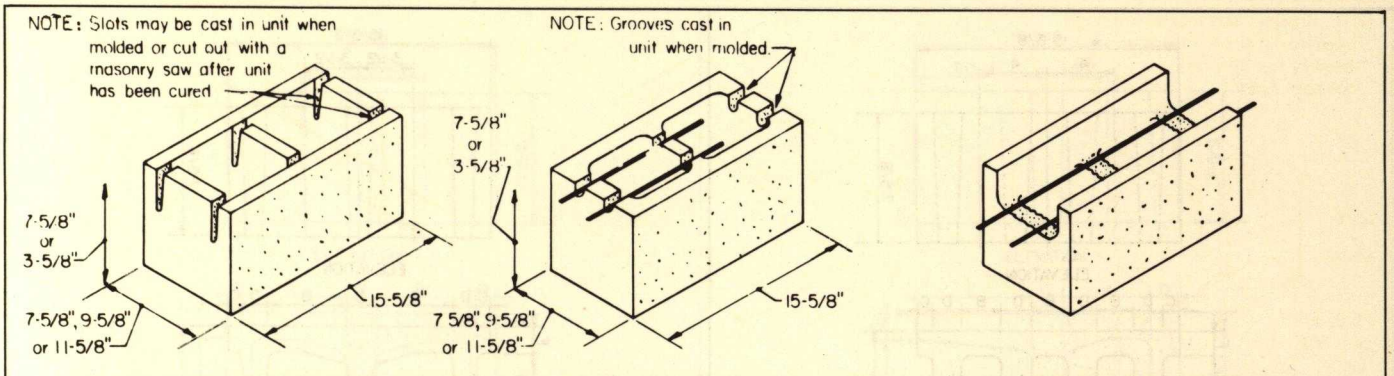

Vertical reinforcement. Set and tie in position after first course has been laid. Knockout ends of block units as required to fit around vertical bars in place.
Pea gravel concrete or grout core-fill in bond beams and reinforced vertical cells. Place as wall is laid up. Maximum height of pour not to exceed 4 feet.
Place metal lath or wire screen in mortar joint under bond beams courses over cores of unreinforced vertical cells to prevent filling with concrete or grout.
Horizontal bond beam reinforcement. Set in place in bond beams as wall is laid up.
Basement floor slab.
Horizontal bond beam in top course and intermediate courses as required by the design. See Detail 1. for typical bond beam unit.
Footing.
Mortar cross webs adjacent to vertically reinforced and filled cells to prevent leakage of concrete or grout into adjacent cells.

DETAIL 2. TYPICAL REINFORCED CONCRETE MASONRY WALL CONSTRUCTION ~ REINFORCEMENT AND CORE-FILL PLACED AS WALL IS LAID UP.


Prefabricated trussed-type horizontal joint reinforcement with deformed high-tensile strength steel longitudinal rods in horizontal mortar joints at spacing as required.

DETAIL 3. TYPICAL REINFORCED CONCRETE MASONRY CONSTRUCTION USING HORIZONTAL JOINT REINFORCEMENT IN LIEU OF BOND BEAMS TO PROVIDE LATERAL REINFORCEMENT.

Figure 39 - Suggested Construction Details for Reinforced Concrete Masonry Foundation Walls.

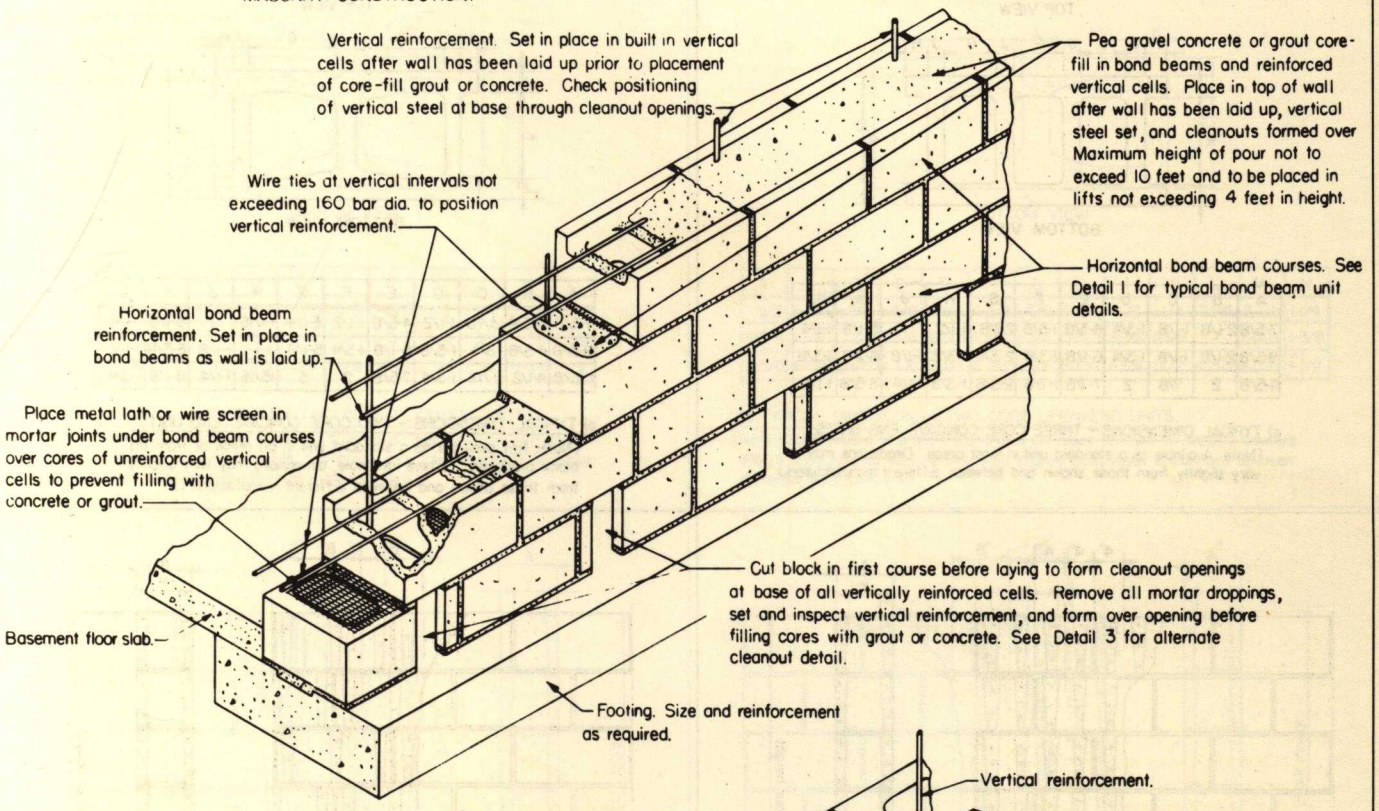

NOTE: Slots may be cast in unit when molded or cut out with a masonry saw after unit has been cured
7-5/8" or 3-5/8"
7-5/8", 9-5/8" or 11-5/8"
15-5/8"
NOTE: Grooves cast in unit when molded.
7-5/8" or 3-5/8"
7 5/8", 9-5/8" or 11-5/8"
15-5/8"

a) Standard units with knockout slots.

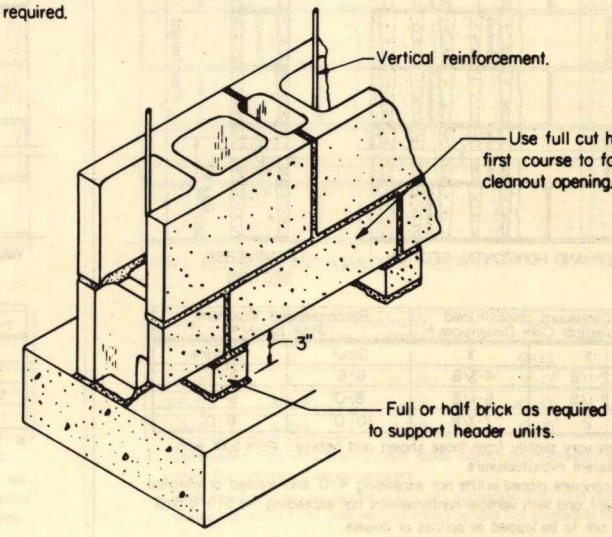
a) Grooved beam unit.

b) Standard unit with sections of end and cross webs removed to permit placement of reinforcing.

DETAIL 1. TYPICAL BOND BEAM UNITS USED IN REINFORCED CONCRETE MASONRY CONSTRUCTION.

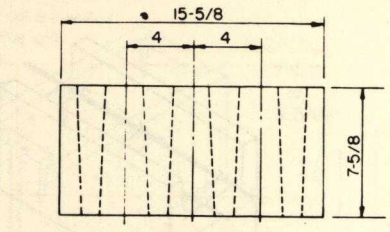

Vertical reinforcement. Set in place in built in vertical cells after wall has been laid up prior to placement of core-fill grout or concrete. Check positioning of vertical steel at base through cleanout openings.
Pea gravel concrete or grout core-fill in bond beams and reinforced vertical cells. Place in top of wall after wall has been laid up, vertical steel set, and cleanouts formed over Maximum height of pour not to exceed 10 feet and to be placed in lifts not exceeding 4 feet in height.
Wire ties at vertical intervals not exceeding 160 bar dia. to position vertical reinforcement.
Horizontal bond beam courses. See Detail 1 for typical bond beam unit details.
Horizontal bond beam reinforcement. Set in place in bond beams as wall is laid up.
Place metal lath or wire screen in mortar joints under bond beam courses over cores of unreinforced vertical cells to prevent filling with concrete or grout.
Cut block in first course before laying to form cleanout openings at base of all vertically reinforced cells. Remove all mortar droppings, set and inspect vertical reinforcement, and form over opening before filling cores with grout or concrete. See Detail 3 for alternate cleanout detail.
Basement floor slab.
Footing. Size and reinforcement as required.

DETAIL 2. TYPICAL REINFORCED CONCRETE MASONRY WALL CONSTRUCTION - VERTICAL REINFORCEMENT AND CORE-FILL PLACED AFTER WALL HAS BEEN LAID UP.

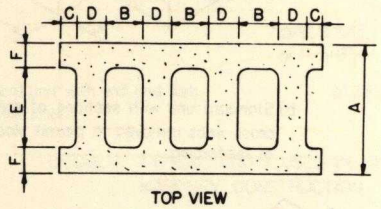

Vertical reinforcement.
Use full cut headers in first course to form continuous cleanout opening.
3"
Full or half brick as required to support header units.

DETAIL 3. ALTERNATE CLEANOUT DETAIL.

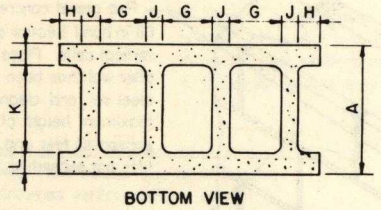
Figure 40 - Suggested Construction Details for Reinforced Concrete Masonry Foundation Walls.



ELEVATION



TOP VIEW

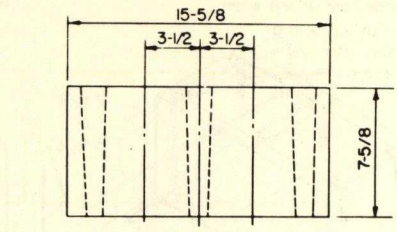


BOTTOM VIEW

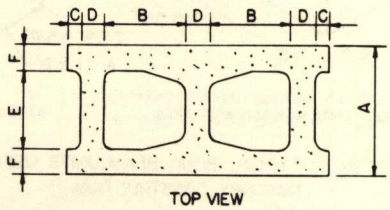
A	B	C	D	E	F	G	H	J	K	L
7-5/8	2-1/8	1-1/8	1-3/4	4-3/8	1-5/8	2-7/8	1-1/2	1	5-1/8	1-1/4
9-5/8	2-1/8	1-1/8	1-3/4	6-1/8	1-3/4	2-3/4	1-3/8	1-1/8	6-7/8	1-3/8
11-5/8	2	7/8	2	7-7/8	1-7/8	2-5/8	1-3/8	1-1/4	8-5/8	1-1/2

a) TYPICAL DIMENSIONS ~ THREE CORE CONCAVE-END UNITS.

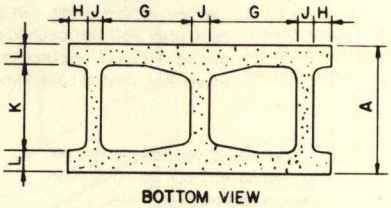
(Note: Available as a standard unit in most areas. Dimensions may vary slightly from those shown and between different manufacturers.)



ELEVATION



TOP VIEW

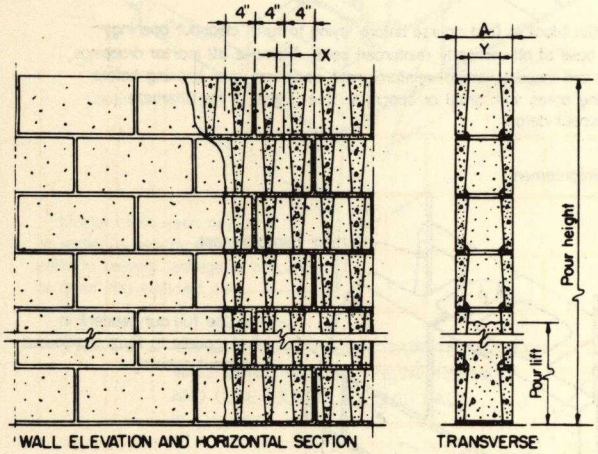


BOTTOM VIEW

A	B	C	D	E	F	G	H	J	K	L
7-5/8	4-3/4	13/16	1-1/2	4-5/8	1-1/2	5-1/4	1-1/16	1	5-1/8	1-1/4
9-5/8	4-5/8	3/4	1-5/8	6-1/8	1-3/4	5-1/8	1	1-1/8	6-5/8	1-1/2
11-5/8	4-1/2	11/16	1-3/4	7-5/8	2	5	15/16	1-1/4	8-1/8	1-3/4

b) TYPICAL DIMENSIONS ~ TWO CORE CONCAVE-END UNITS.

(Note: Not available as a standard unit in all areas or from all block producers. Where available, dimensions may vary slightly from those shown and between different manufacturers.)



WALL ELEVATION AND HORIZONTAL SECTION

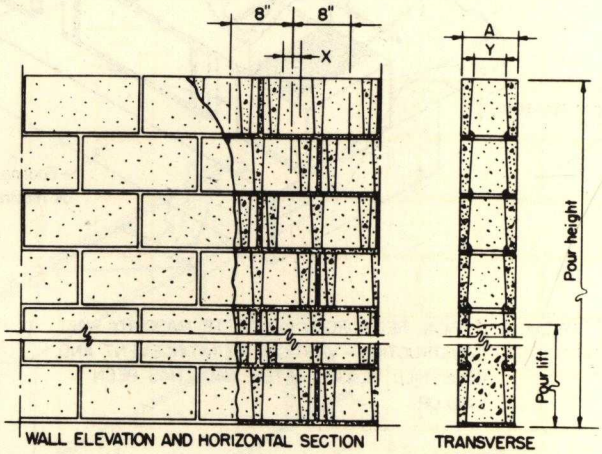
TRANSVERSE SECTION

Wall Thickness	Continuous Unobstructed Vertical Core Dimensions *		Recommended Maximum Pour Height **	
A	X	Y	Grout	Concrete
7-5/8	2-1/8	4-3/8	6'-6"	5'-6"
9-5/8	2-1/8	6-1/8	8'-0"	6'-6"
11-5/8	2	7-7/8	10'-0"	8'-0"

* Dimensions may vary slightly from those shown and between walls built with units from different manufacturers.

** With grout or concrete placed in lifts not exceeding 4'-0" and rodded or vibrated during placement, and with vertical reinforcement not exceeding No.5(5/8"dia.) in size if bars are to be lapped at splices or dowels.

c) TYPICAL WALL CONSTRUCTION WITH THREE CORE CONCAVE-END UNITS.



WALL ELEVATION AND HORIZONTAL SECTION

TRANSVERSE SECTION

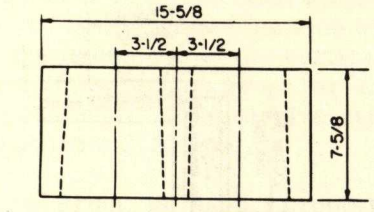
Wall Thickness	Continuous Unobstructed Vertical Core Dimensions *		Recommended Maximum Pour Height **	
A	X	Y	Grout	Concrete
7-5/8	3	4-5/8	7'-6"	6'-0"
9-5/8	2-7/8	6-1/8	9'-0"	7'-6"
11-5/8	2-3/4	7-5/8	10'-0"	8'-6"

* Dimensions may vary slightly from those shown and between walls built with units from different manufacturers.

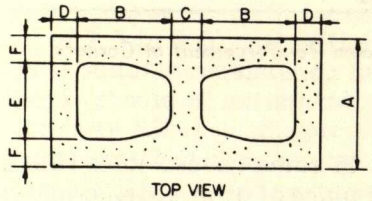
** With grout or concrete placed in lifts not exceeding 4'-0" and rodded or vibrated during placement, and with vertical reinforcement not exceeding No.6 (3/4"dia.) in size if bars are to be lapped at splices or dowels.

d) TYPICAL WALL CONSTRUCTION WITH TWO CORE CONCAVE-END UNITS.

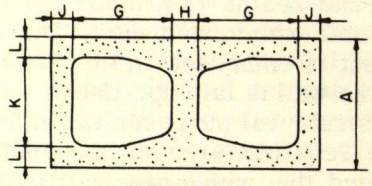
Figure 41 - Typical Dimensions of Three and Two Core Concave-End Units and Wall Constructions.



ELEVATION



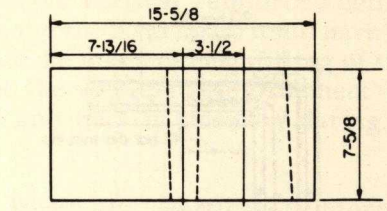
TOP VIEW



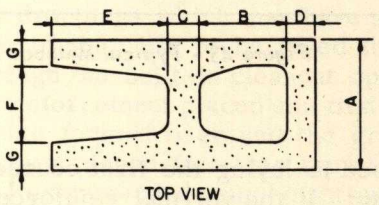
BOTTOM VIEW

A	B	C	D	E	F	G	H	J	K	L
7-5/8	5-1/4	1-7/8	1-5/8	4-3/8	1-5/8	5-13/16	1-1/2	1-1/4	5-1/8	1-1/4
9-5/8	5-1/16	2	1-3/4	6-1/8	1-3/4	5-5/8	1-5/8	1-3/8	6-7/8	1-3/8
11-5/8	4-7/8	2-1/8	1-7/8	7-7/8	1-7/8	5-7/16	1-3/4	1-1/2	8-5/8	1-1/2

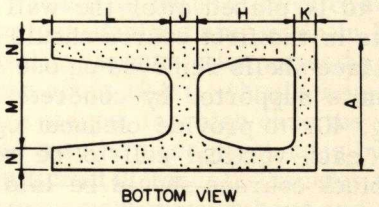
a) TYPICAL DIMENSIONS ~ TWO CORE PLAIN-END UNITS.
(Note: Not available as a standard unit in all areas or from all block producers. Where available, dimensions may vary slightly from those shown and between different manufacturers.)



ELEVATION



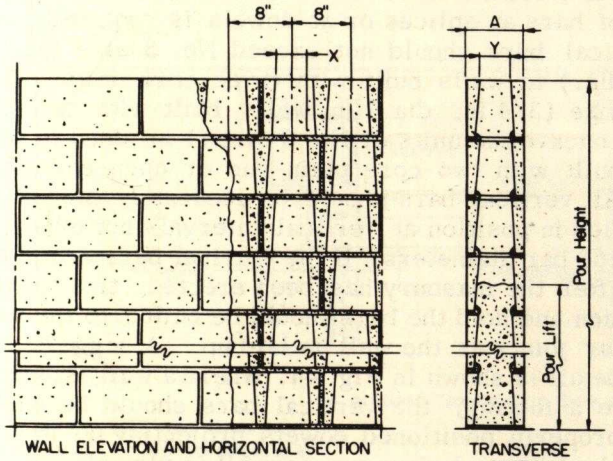
TOP VIEW



BOTTOM VIEW

A	B	C	D	E	F	G	H	J	K	L	M	N
7-5/8	5-1/4	1-7/8	1-5/8	6-7/8	4-3/8	1-5/8	5-13/16	1-1/2	1-1/4	7-1/16	5-1/8	1-1/4
9-5/8	5-1/16	2	1-3/4	6-13/16	6-1/8	1-3/4	5-5/8	1-5/8	1-3/8	7	6-7/8	1-3/8
11-5/8	4-7/8	2-1/8	1-7/8	6-3/4	7-7/8	1-7/8	5-7/16	1-3/4	1-1/2	6-15/16	8-5/8	1-1/2

b) TYPICAL DIMENSIONS ~ TWO CORE OPEN-END UNITS.
(Note: Not available as a standard unit in all areas or from all block producers. Where available, dimensions may vary slightly from those shown and between different manufacturers.)



WALL ELEVATION AND HORIZONTAL SECTION

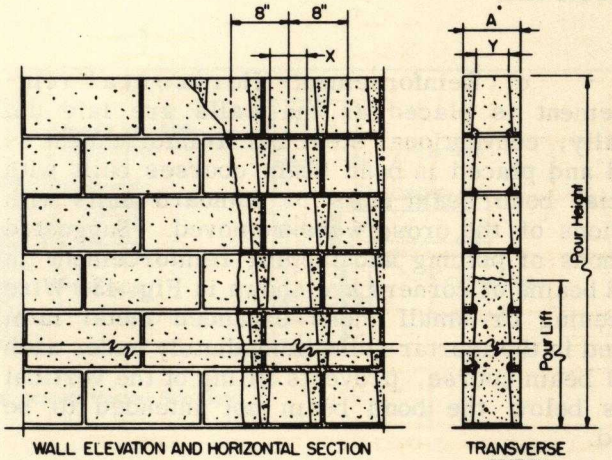
TRANSVERSE SECTION

Wall Thickness	Continuous Unobstructed Vertical Core Dimensions *		Recommended Maximum Pour Height **	
A	X	Y	Grout	Concrete
7-5/8	4-3/8	4-3/8	8'-6"	7'-0"
9-5/8	4-1/8	6-1/8	10'-0"	8'-0"
11-5/8	3-7/8	7-7/8	10'-0"	10'-0"

* Dimensions may vary slightly from those shown and between walls built with units from different manufacturers.

** With grout or concrete placed in lifts not exceeding 4'-0" and rodded or vibrated during placement and with vertical reinforcement not exceeding No. 8 (1" dia.) in size.

c) TYPICAL WALL CONSTRUCTION WITH TWO CORE PLAIN-END UNITS.



WALL ELEVATION AND HORIZONTAL SECTION

TRANSVERSE SECTION

Wall Thickness	Continuous Unobstructed Vertical Core Dimensions *		Recommended Maximum Pour Height **	
A	X	Y	Grout	Concrete
7-5/8	5	4-3/8	9'-0"	7'-6"
9-5/8	4-7/8	6-1/8	10'-0"	9'-0"
11-5/8	4-3/4	7-7/8	10'-0"	10'-0"

* Dimensions may vary slightly from those shown and between walls built with units from different manufacturers.

** With grout or concrete placed in lifts not exceeding 4'-0" and rodded or vibrated during placement and with vertical reinforcement not exceeding No. 8 (1" dia.) in size.

d) TYPICAL WALL CONSTRUCTION WITH TWO CORE OPEN-END UNITS.

Figure 42 - Typical Dimensions of Two Core Plain-End and Open-End Units and Wall Constructions.

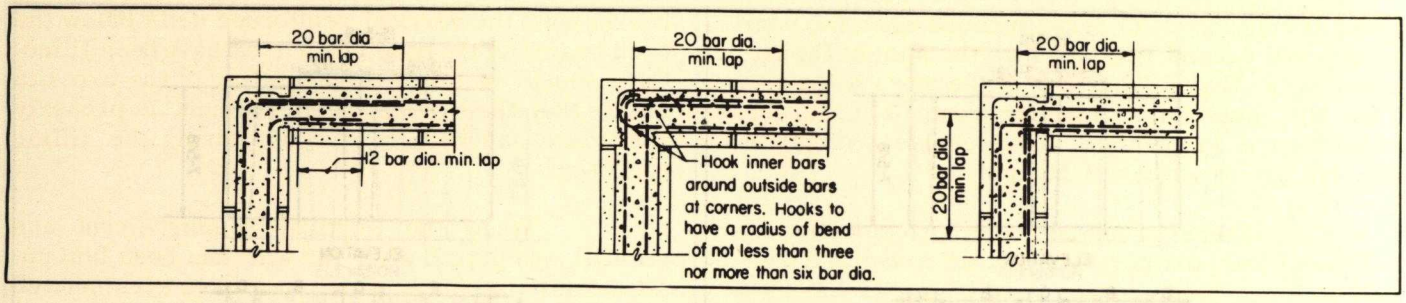


Figure 43 - Typical Methods of Placing Horizontal Bond Beam Reinforcement at Corners.

should be used in laying the first course of block on the footing. If the vertical reinforcement and corefill are to be placed after the wall has been laid up, units in the first course should have portions of the face shells removed on one side or be of header units supported by concrete brick, as shown in Fig. 40, to provide cleanout openings at the base of each vertical cell to be reinforced. Remaining block courses should be laid with face shell mortar bedding in accordance with accepted standards of good workmanship. Cross webs adjacent to vertical cells to be filled should be fully bedded in mortar to prevent leakage of concrete or grout during the filling operation. Except where corefill and block masonry are carried up simultaneously, mortar joints on the interior of cells to be filled should be struck flush by the mason as each course is laid to remove mortar protruding into the cells which would impede the flow of grout or concrete fill.

c. Reinforcement--Horizontal reinforcement is placed as the walls are laid up. Usually, conventional steel bar reinforcement is used and placed in bond beam courses built with special bond beam units or standard units with portions of the cross webs removed. Suggested methods of placing and lapping reinforcement in bond beams at corners are shown in Fig. 43. Wire screening or small mesh expanded metal lath, placed in the mortar joint immediately under each bond beam course, prevents filling of the vertical cells below the bond beam not intended to be filled.

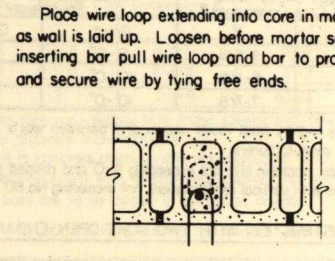

Place wire loop extending into core in mortar joints as wall is laid up. Loosen before mortar sets. After inserting bar pull wire loop and bar to proper position and secure wire by tying free ends.

Figure 44 - Typical Detail of Tie to Hold Vertical Reinforcement in Place While Cell is Filled.

In some cases, joint reinforcement may be used in place of or in conjunction with reinforcement in bond beams to provide the required area of horizontal reinforcement. This type of reinforcement is embedded in the horizontal mortar joints as the wall is laid up. Use of joint reinforcement for structural purposes is limited, however, since the required area of horizontal steel will often exceed that which can be provided by No. 6 gage wire, the largest size which can be accomodated in a standard 3/8 in. mortar joint. Where joint reinforcement is used, a prefabricated type made from high tensile strength, deformed, cold-drawn steel wire is recommended.

To insure complete embedment of vertical reinforcement, a minimum clearance of 1/2 in. is recommended between the vertical bars and the closest side of the reinforced cores. If lapping of bars at splices or at dowels is required, vertical bars should not exceed No. 5 size (5/8 in. dia.) in walls built with three-core units, No. 6 size (3/4 in. dia.) in walls built with two-core concave end units or No. 8 size (1 in. dia.) in walls built with two-core plain-end or open-end units. All vertical bars should be accurately placed and tied in position at vertical intervals not exceeding 160 bar diameters. If the vertical bars are placed after the masonry has been erected, ties to position and hold the bars should be built into the mortar joints as the wall is laid up. A suggested tie detail is shown in Fig. 44. Where a wall is doweled to a footing, the vertical bars should be tied to properly positioned dowels projecting up into the block cores before any corefill is placed.

d. Mortar--Mortar for reinforced concrete masonry foundation walls should comply with the requirements for Type M (A-1) as shown in Table 2. Some codes require that the mortar for such construction have a minimum compressive strength of 2,000 psi, or somewhat less than the 2,500 psi value expected of Type M (A-1) mortar.

e. Corefill--Either a portland cement and coarse sand grout or a pea-gravel concrete may be used to fill the reinforced cells. As a

general rule, grout is recommended when the pour height will exceed twelve times the sum of the unobstructed clear dimensions of the vertical cavity. The fill material, whether grout or concrete, should have an ultimate compressive strength at 28 days age of at least 2,500 psi.

Where grout is used, proportions by volume of one part portland cement to two and one-half to three parts coarse sand (fineness modulus within the range of 2.5 to 3.5) and sufficient water to produce a mix of suitable consistency for pouring without segregation should be satisfactory. Where a pea-gravel concrete fill is used, proportions by volume of one part portland cement, two and one-half parts concrete sand, and two parts pea-gravel aggregate with sufficient water to produce a slump of 6 to 8 in., are suggested. Maximum size of coarse aggregate should not exceed 1/2 in. The use of air-entraining portland cement or an air-entraining admixture with regular cement is recommended to improve the flowability of the grout or concrete.

f. Placing Corefill--When vertical reinforcement is set and cores filled as the wall is laid up, the wall construction should not be carried up to heights exceeding about 4 ft. prior to placement of fill. The grout or concrete fill should be rodded or vibrated when placed to insure complete filling of the cavities. If the time interval between pours will exceed one hour, the pour should be stopped at least 1-1/2 in. below a mortar joint. Construction should not proceed above a bond beam course until the vertical reinforced cells below the bond beam and the bond beam itself have been filled. Care should be taken during filling of the cores to insure that the vertical reinforcement is properly positioned and held in position during the filling operation.

When the vertical reinforcement and corefill are placed after the wall has been laid up, all mortar droppings which may have accumulated at the base of the cells to be filled should be removed through the built-in cleanout openings, the vertical reinforcement placed and tied in position, the cleanouts formed over and the grout or concrete fill then placed from the top of the wall in one pour. Recommended pour heights when placing corefill in walls built with three-core and two-core units of various designs are given in Figs. 41 and 42. Generally, construction extending to heights greater than about 10 ft., should be completed in two or more stages, in which case the vertical reinforcement may be made continuous by welding or lapping a sufficient distance to transfer the stresses in the reinforcement through bond and shear. The fill should be placed in lifts not exceeding 4 ft., rodded or vibrated during placement, and at least 15 minutes allowed between successive lifts to permit settlement.

3. Anchorage

Basement walls designed to resist lateral earth pressure through bending in the vertical span must be connected or anchored to the first floor

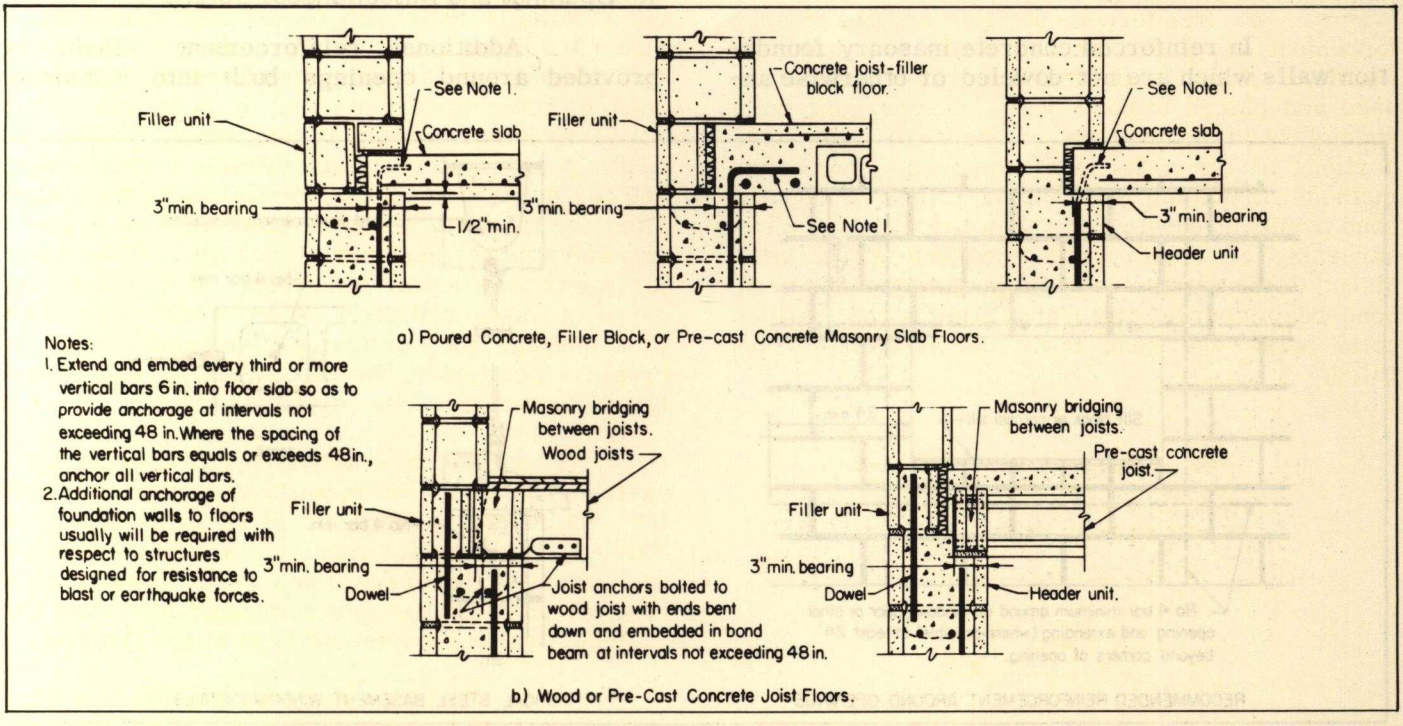

See Note 1.
Filler unit
Concrete slab
3"min. bearing
1/2"min.
Concrete joist-filler block floor.
See Note 1.
3"min. bearing
Header unit
a) Poured Concrete, Filler Block, or Pre-cast Concrete Masonry Slab Floors.
Notes:
1. Extend and embed every third or more vertical bars 6 in. into floor slab so as to provide anchorage at intervals not exceeding 48 in. Where the spacing of the vertical bars equals or exceeds 48 in., anchor all vertical bars.
2. Additional anchorage of foundation walls to floors usually will be required with respect to structures designed for resistance to blast or earthquake forces.
Masonry bridging between joists.
Wood joists
Filler unit
3"min. bearing
Dowel
Joist anchors bolted to wood joist with ends bent down and embedded in bond beam at intervals not exceeding 48 in.
Pre-cast concrete joist.
Header unit.
b) Wood or Pre-Cast Concrete Joist Floors.

Figure 45 - Typical Foundation Wall - Floor Framing and Connection Details.

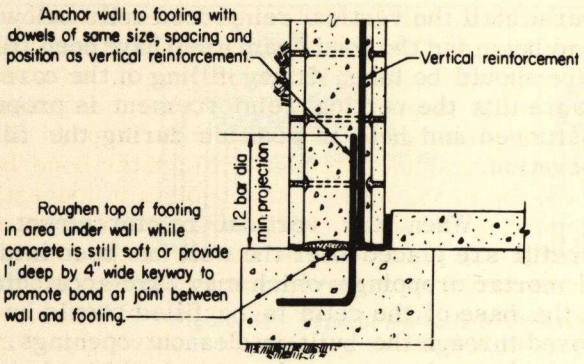

Anchor wall to footing with dowels of same size, spacing and position as vertical reinforcement.
Vertical reinforcement
12 bar dia. min. projection
Roughen top of footing in area under wall while concrete is still soft or provide 1" deep by 4" wide keyway to promote bond at joint between wall and footing.

Figure 46 - Typical Foundation Wall - Footing Connection Where Basement Slab is Not Designed to Resist Inward Wall Reaction.

construction so that the top of the wall is restrained against both outward and inward movement. If the first floor construction is relatively heavy, as in the case of concrete slab floors, special anchorage may not be required, the weight of the floor bearing on the wall providing sufficient restraint. With light floor constructions, or where the foundation walls are required to resist high lateral forces, it may be necessary to provide special anchorage. Suggested first floor connection or bearing details are shown in Fig. 45.

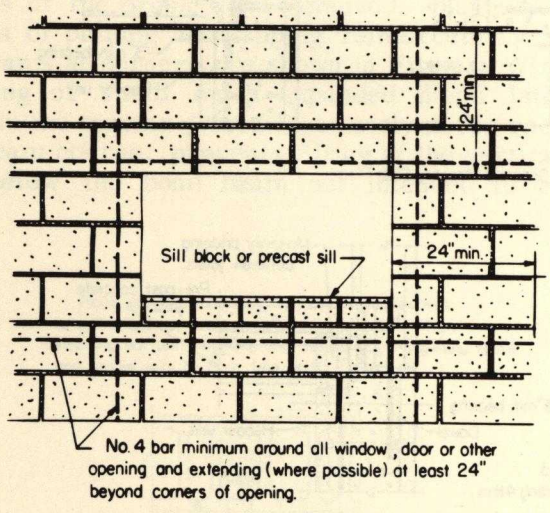
Similarly, where there is no basement floor slab or where the floor slab cannot be considered as adequate support against lateral movement of the base of foundation walls, it will be necessary to anchor the wall to the footing by means of dowels set and cast in the footing and projecting up into the reinforced cores in the wall. A suggested detail is shown in Fig. 46.

In reinforced concrete masonry foundation walls which are not doweled or otherwise anchored to the lateral supporting members at the top and bottom, a portion of the vertical reinforcement should be extended 6 in. past the face or into the supports, or have hooked ends to meet the usual anchorage requirements for reinforcement in simply supported members. If the spacing of the vertical reinforcement is less than 48 in. o.c., one-third or more of the bars should be anchored so as to provide anchorage at intervals not exceeding 48 in. o.c. If the spacing of the vertical reinforcement will equal or exceed 48 in., every bar should be anchored.

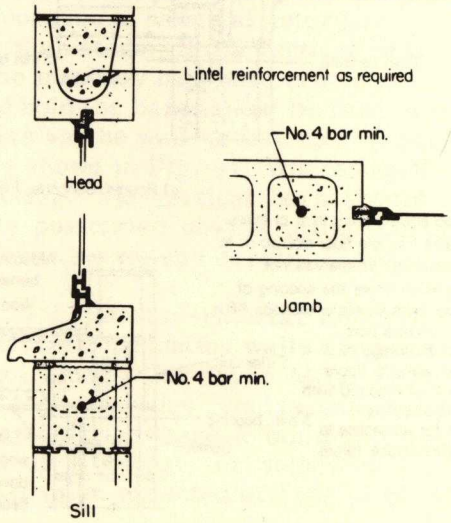
Intersecting bearing walls and nonbearing partitions, if reinforced, should be tied together at intersections with metal ties or anchors embedded in filled cavities in the two walls, by extending and lapping the horizontal reinforcement in the intersecting wall into the other a distance sufficient to provide continuity at the intersection, or by other approved means. Suggested methods of placing and lapping reinforcement in bond beams at corners have been shown in Fig. 43, and similar details may be employed at other intersections. Metal bar or loop type anchors are sometimes used where bond beams fall at different levels or where it is not feasible to make bond beams continuous across the intersection. In such cases, the anchors should be of a size and installed at spacings as required to develop the full strength of the walls at the intersection. At intersections of unreinforced and reinforced walls, the anchorage details recommended in the section on plain masonry are applicable.

4. Openings and Miscellaneous Details

Additional reinforcement should be provided around openings built into reinforced


24" min.
Sill block or precast sill
24" min.
No. 4 bar minimum around all window, door or other opening and extending (where possible) at least 24" beyond corners of opening.

RECOMMENDED REINFORCEMENT AROUND OPENINGS



TYPICAL STEEL BASEMENT WINDOW DETAILS

Figure 47 - Typical Reinforcing and Window Detail at Openings in Reinforced Concrete Masonry Basement Walls.

concrete masonry walls. Such additional reinforcement should consist of not less than one 1/2 in. diameter bar above, below, and on each side of the opening and extending at least 24 in. past the corners as shown in Fig. 47. If properly designed bond beams are provided immediately above and below openings, additional horizontal reinforcement at these locations will not ordinarily be required.

The recommendations given on the use of expansion and control joints in unreinforced concrete masonry foundation walls also apply to reinforced concrete masonry construction. Generally, there will be little need for expansion joints in walls under 150 to 200 ft. in length or for control joints in walls under 100 ft. in length. If used in long walls supporting wood frame construction, control joints generally should be stopped at the bottom of and not extended through the bond beam at the top of the wall. Control joints in foundations supporting masonry walls may be extended up through the bond beam at the top of the foundation and control joints provided at the same point in the walls above grade. In all other cases, control joints in the above grade walls should be stopped at the top of, and not extended through, the bond beam at the top of the foundation walls.

VI. WATERPROOFING, CONDENSATION AND TERMITE PROTECTION

Of the various problems encountered in the design of foundation walls, those relating to the need for special measures or construction to prevent penetration of moisture from the ground into or through the walls, to minimize the possible occurrence of condensation on the inner surface of basement walls, and to protect wood members bearing on foundations from attack by subterranean termites are of particular importance. Generally speaking, waterproofing and condensation control problems usually are associated with basement walls since basements, to serve satisfactorily as storage, living, or service areas, must be maintained dry. In any particular case, the need for and extent to which special measures to prevent moisture penetration and condensation may be required will be dependent upon a number of factors, including soil and climatic conditions, the type and quality of the construction, and the importance of the structure. Similarly, the need for special construction or other measures to protect against subterranean termite attack will vary in different areas. Although no precise geographical limits delineating areas of minimum and maximum termite hazard can be given, it is definitely of greatest magnitude in the southern latitudes.

Recommended methods of waterproofing concrete masonry foundations, a brief discussion of suggested methods of controlling condensation in basements, and suggested methods of capping concrete masonry foundation walls to provide an effective termite barrier are given in the following for various assumed conditions and degrees of exposure. Economic considerations usually will require that the measures employed in any particular case bear some relationship to the severity of the hazard in question. Accordingly, the designer should determine those assumed conditions or degree of hazard which most closely approximates actual or local conditions and apply the methods or construction recommended for such conditions.

It is believed that the recommendations given in the following will provide the necessary protection consistent with the degree of hazard assumed. However, they may not comply in all respects with arbitrary minimum requirements of local building codes, governmental, or other regulatory bodies. Therefore, it is suggested that designers check and insure that the methods or construction selected will comply with such local requirements before adopting or incorporating them in the design.

A. WATERPROOFING

1. Surface and Subsurface Drainage

Regardless of wall design, soil conditions or location, it is good practice to minimize the accumulation of surface and ground water adjacent to foundations, especially basement walls. Wherever possible, the finish grade should be sloped to drain surface water away from the foundations. Tile drains or ditches may be required to intercept and divert the rapid run-off from hillside locations. The discharge from downspouts should be directed away from the foundation.

A tile drain extending around the footings of basement walls is generally recommended in areas with wet climates or in wet and poorly drained soil. Such tile drains must be connected to a suitable outlet such as a surface drainage channel, separate storm water sewer, or a dry well located some distance away from the structure. Connection to a public sewer should not be made without first obtaining the necessary permit or approval from the proper official. Under some conditions, not ordinarily encountered, it is necessary to connect the tile drain to a sump from which the discharge may be pumped into a drainage channel at some higher elevation.

Subsurface drainage systems around residences and buildings consist of clay or concrete tile, usually of 4 in. inside diameter, laid with 1/8 in. open joints, and with a gradient of fall of 4 in. or more per 100 ft. The tile line should be built close to the foundation as shown in Fig. 48 on page 58, but, never where it might disturb or undermine the

soil under the footing. To prevent fine soil from entering the drain, joints between tile should be covered on top with a strip of roofing felt, and coarse gravel or crushed stone filled around and over the tile to a depth of 12 in. or more.

Tile drains are less essential and often are omitted in areas of low rainfall or where the soil is effectively self-draining. Past experience with similar construction is perhaps the best criterion as to the need for tile drains in a particular location.

2. Waterproof Coatings

While few masonry coatings are entirely impervious to water, the ones recommended and commonly used are "waterproof" in the sense that when properly applied they prevent perceptible moisture penetration into or through even relatively permeable masonry.

The more common and successfully used types of coatings are the following:

a) Portland cement-sand plaster with or without admixtures such as iron filings, hydrated lime and water repellants. Such plaster is applied in one or more coats, each about 1/4 in. thick. The mortar mixes recommended for foundation work may also be used as plaster.

b) Bituminous (tar or asphalt) coatings. These include brushed or mopped-on coatings of hot tar or asphalt; sprayed or brushed-on coatings of cold cut-back asphalts or asphalt emulsions; and thick troweled-on coatings of cold, cut-back asphalts or asphalt emulsions of trowel consistency and with or without asbestos fiber reinforcement.

c) Bituminous membranes consisting of alternate layers of either hot or cold applied bituminous materials and asphalt impregnated felt paper, cotton cloth specially treated to resist rot and mildew, or glass cloth.

Other types of coatings, sometimes but less commonly used for waterproofing foundations include portland cement paints and grout coats and so-called "plastic" coatings.

In new construction, waterproof coatings should always be applied to the exterior or earth side of basement walls. Since, in this location, they are not accessible for maintenance and repair except at great expense, the durability of waterproof coatings is an important consideration. The coating must remain in place, bond to the masonry, and retain its effectiveness for many years, preferably for the ordinary life of the structure.

Regardless of type or composition, waterproofing materials will be effective only insofar as they form a continuous coating from some point above finish grade to a point below the top of the footing. The type of coating selected for use in any particular case and the application methods employed must be designed to insure satisfactory adhesion of the coating to the masonry and complete sealing of the surface, including especially the surface voids of coarse textured units. Proper preparation and cleaning of the wall surface are of primary importance with all types of coatings. With portland cement plaster and similar cement coatings, dampening of the wall surfaces prior to and adequate moist curing subsequent to application are essential to insure adequate strength gain and bond to the wall surfaces. On the other hand, bituminous coatings containing petroleum solvents, such as cut-back tars and asphalts, should be applied only to dry wall surfaces and, accordingly, sufficient time must be allowed after walls have been laid up for the wall surfaces to air dry before application of such coatings. Asphaltic emulsions generally will bond satisfactorily to either damp or dry wall surfaces, however, they are water soluble when applied and should be protected from contact with free water or ground moisture following application until they have cured and hardened to the point where they are not water soluble. Ordinarily, this will be between 60 to 72 hours following application. Particular attention should be given to the elimination of pin holes or breaks in the film when applying bituminous coatings by spray, brush, or mopping. Troweled-on coatings should be applied with sufficient pressure to insure that the mastic will be forced into and seal all surface voids and bond to the wall surface.

3. Foundation Walls (not enclosing excavated areas)

a) When built of good quality, low absorption concrete units, ASTM Type S or stronger mortar, and tightly tooled or pointed joints, foundations not enclosing basement areas will not require special waterproofing measures except where subject to severe frost action. Where subject to severe frost action, the application of two coats of bituminous waterproofing to exterior surfaces below grade is recommended.

b) When built of porous, open-textured units, it is recommended that both faces of the walls be protected with one 1/4 in. thick coat of portland cement plaster. If subject to severe frost action, two coats of bituminous waterproofing is recommended on the exterior face in addition to the portland cement plaster.

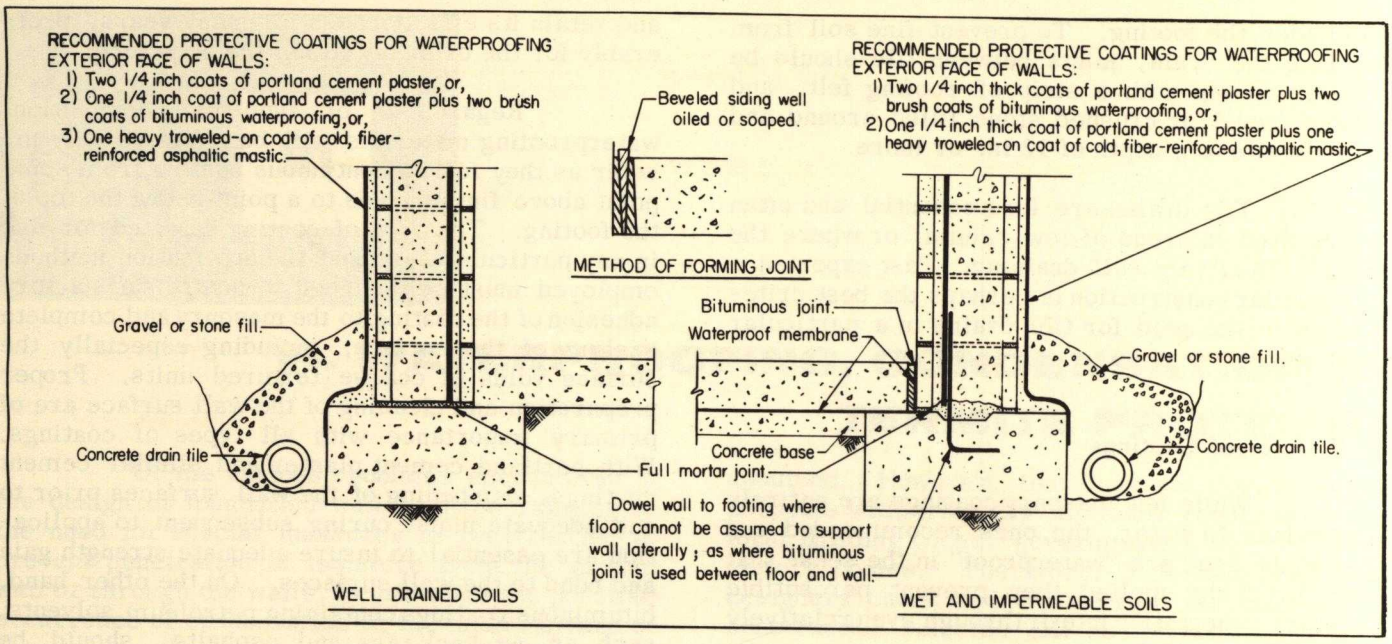

RECOMMENDED PROTECTIVE COATINGS FOR WATERPROOFING EXTERIOR FACE OF WALLS:
1) Two 1/4 inch coats of portland cement plaster, or,
2) One 1/4 inch coat of portland cement plaster plus two brush coats of bituminous waterproofing, or,
3) One heavy troweled-on coat of cold, fiber-reinforced asphaltic mastic.
Gravel or stone fill.
Concrete drain tile
WELL DRAINED SOILS
Beveled siding well oiled or soaped.
METHOD OF FORMING JOINT
Bituminous joint.
Waterproof membrane
Concrete base
Full mortar joint.
Dowel wall to footing where floor cannot be assumed to support wall laterally; as where bituminous joint is used between floor and wall.
RECOMMENDED PROTECTIVE COATINGS FOR WATERPROOFING EXTERIOR FACE OF WALLS:
1) Two 1/4 inch thick coats of portland cement plaster plus two brush coats of bituminous waterproofing, or,
2) One 1/4 inch thick coat of portland cement plaster plus one heavy troweled-on coat of cold, fiber-reinforced asphaltic mastic.
Gravel or stone fill.
Concrete drain tile.
WET AND IMPERMEABLE SOILS

Figure 48 - Suggested Methods of Waterproofing Concrete Masonry Basement Walls.

4. Basement Walls (Fig. 48)

a) When built in well drained soils, it is recommended that the exterior face of the walls be protected with either:

1) Two 1/4 in. thick coats of portland cement plaster, or

2) One 1/4 in. thick coat of portland cement plaster plus two brush coats of bituminous waterproofing, or

3) One heavy troweled-on coat of cold, fiber-reinforced asphaltic mastic.

b) When built in wet, poorly drained soils, it is recommended that the exterior face of basement walls be protected with either:

1) Two 1/4 in. thick coats of portland cement plaster plus two brush coats of bituminous waterproofing, or

2) One 1/4 in. thick coat of portland cement plaster, plus one heavy troweled-on coat of cold, fiber-reinforced asphaltic mastic.

B. CONDENSATION

Damp or wet basement conditions are often mistakenly attributed to leakage of water through the exterior walls when the trouble actually is due to condensation of moisture from the air inside the basement on the inside wall surfaces. Condensation in basements is most frequently encountered and troublesome in areas having a warm humid climate but can occur wherever the temperature and relative humidity of the air in a basement is maintained at a high level either artificially or due to atmospheric conditions. Essentially, condensation may occur any time the wall surface temperature is below the dew point temperature of the air in the basement. A typical calculation of the relative humidity level above which condensation could occur in a basement surrounded by 8 in. poured concrete walls in one case and 8 in. concrete masonry walls in the other, for the same air and ground temperature, is given in Fig. 49. Note that due to the better insulating value of the concrete masonry, the critical relative humidity level is at about 85% and 78% with the lightweight and heavyweight concrete masonry walls respectively, whereas it falls at about 69% with the poured concrete walls. In this case, therefore, condensation is less likely to occur with the concrete masonry than with poured concrete construction.

Very often condensation in basements can be prevented or minimized by regulating the ventilation of the basement area. In general, basement windows should be kept closed at all times when the heating system is operating; also in the spring months until warm weather has appreciably raised ground and basement wall temperatures. Proper venting of gas hot water heaters, clothes dryers or other appliances tending to release moisture vapor is also helpful. In areas with

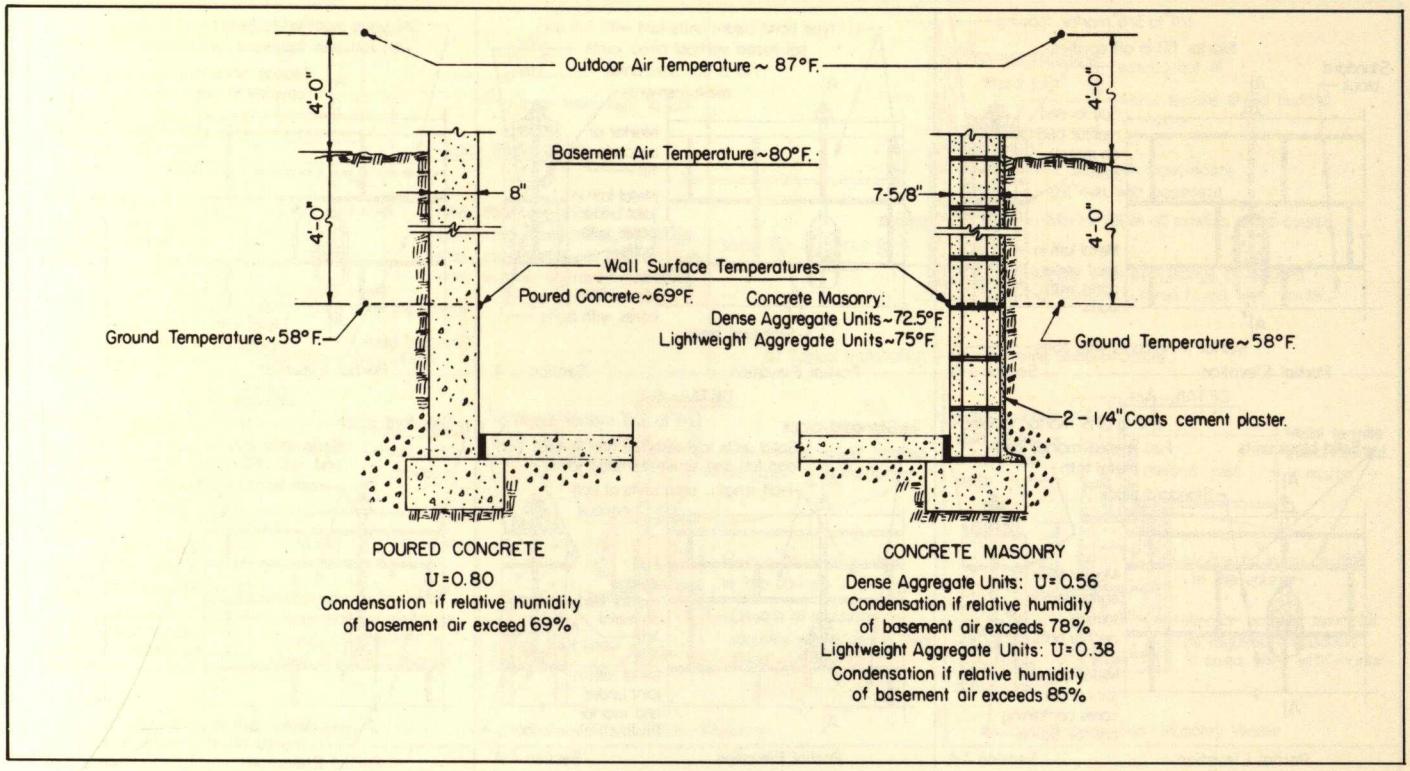

Outdoor Air Temperature ~ 87°F.
4'-0"
4'-0"
Basement Air Temperature ~ 80°F.
8"
Wall Surface Temperatures
Poured Concrete ~ 69°F.
Ground Temperature ~ 58°F.
7-5/8"
Concrete Masonry:
Dense Aggregate Units ~ 72.5°F.
Lightweight Aggregate Units ~ 75°F.
Ground Temperature ~ 58°F.
2 - 1/4" Coats cement plaster.
POURED CONCRETE
U = 0.80
Condensation if relative humidity of basement air exceed 69%
CONCRETE MASONRY
Dense Aggregate Units: U = 0.56
Condensation if relative humidity of basement air exceeds 78%
Lightweight Aggregate Units: U = 0.38
Condensation if relative humidity of basement air exceeds 85%

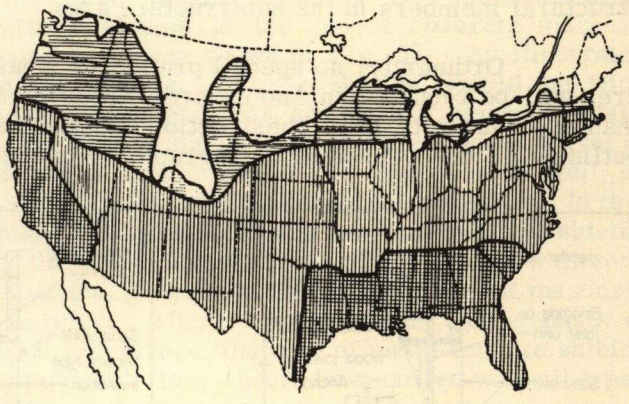
Figure 49 - Comparison of Insulating Value of 8 in. Poured Concrete and 8 in. Concrete Masonry Basement Walls.

warm humid climates, it is sometimes necessary to install a dehumidifier to lower the relative humidity in the basement, in which case introduction of outdoor air into the basement should be avoided or reduced to a minimum.

C. TERMITE PROTECTION

Various measures have been developed to protect wood frame construction from attack by subterranean and other types of termites. These measures include special capping courses for unit masonry foundation walls; chemical treatment of the soil adjacent to foundations, piers, posts, porches, and under slabs on grade; the use of chemically treated or naturally durable woods; and the installation of corrosion resistant metal termite shields on top of poured concrete and unit masonry foundation walls. Generally, the extent to which these protective measures are employed should be based on the severity of the termite hazard in the area in which the construction will be located. In this discussion, it is assumed that any particular area will fall within one of four termite hazard area classifications, namely: (1) no hazard, (2) light hazard, (3) moderate hazard, and (4) severe hazard areas. The determination of the classification within which a particular area or location will fall is best based on local experience or on the recommendations of state entomological authorities. Where such information is not available, Fig. 50 may serve as a guide.

Regardless of the degree of termite hazard, there are certain construction practices which, if followed, will minimize the possibility of termite attack and the need for special protective measures. These include adequate drainage for the site and building; provision of adequate flashing at heads and sills of openings, at gable ends, gravel stops and drip edges; proper ventila-



Region I - No Hazard.

Region II - No Hazard to Light Hazard.

Region III - Light to Moderate Hazard.

Region IV - Moderate to Severe Hazard.

Note: Local conditions may be more or less severe than indicated by the region classification. Such known local conditions should take precedence in determining the applicability of protective measures.

Figure 50 - Geographic Distribution of Termites.

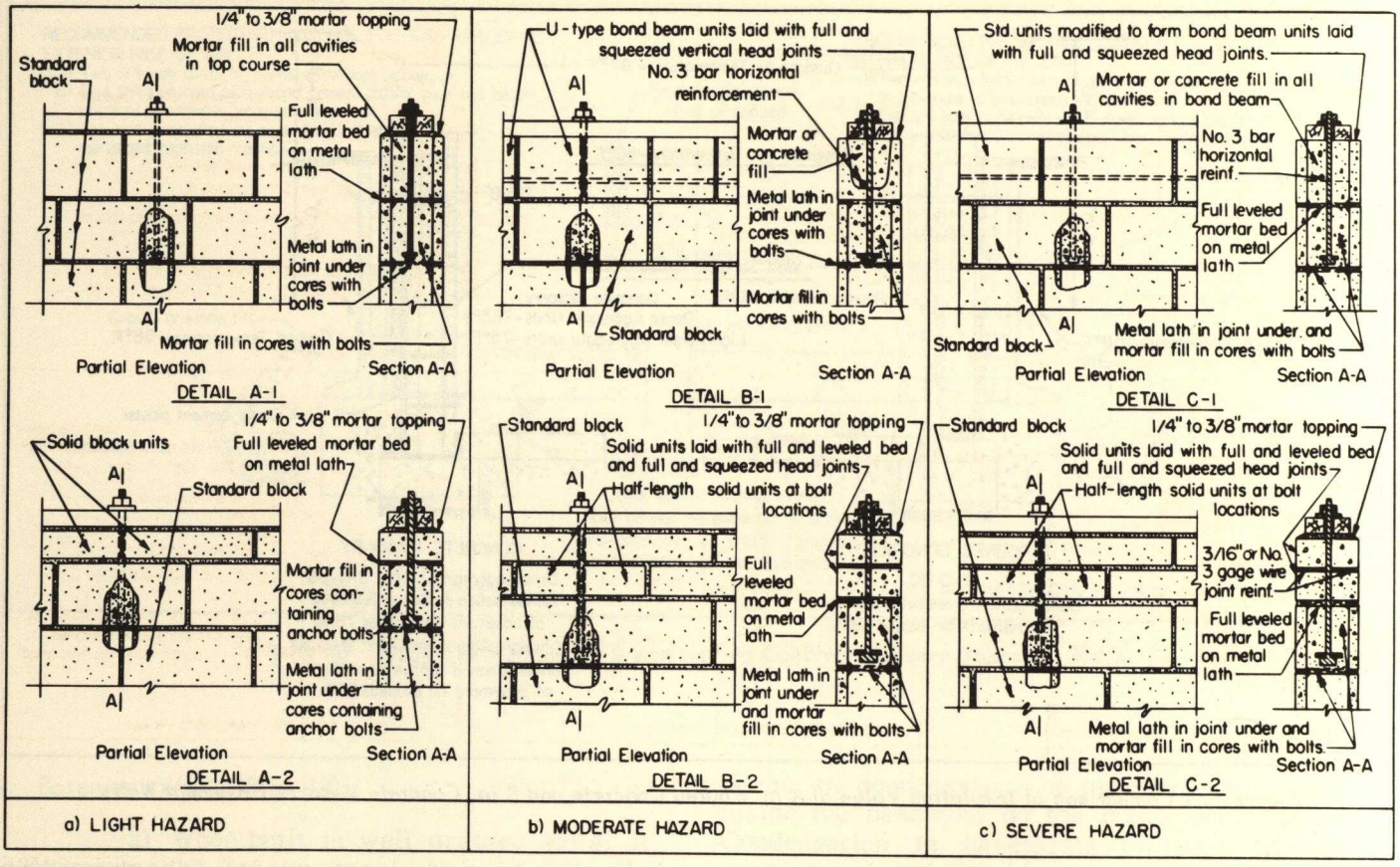

1/4" to 3/8" mortar topping
Mortar fill in all cavities in top course
Standard block
Full leveled mortar bed on metal lath
Metal lath in joint under cores with bolts
Mortar fill in cores with bolts
Partial Elevation
Section A-A
DETAIL A-1
Solid block units
Full leveled mortar bed on metal lath
Standard block
Mortar fill in cores containing anchor bolts
Metal lath in joint under cores containing anchor bolts
DETAIL A-2
U-type bond beam units laid with full and squeezed vertical head joints
No. 3 bar horizontal reinforcement
Mortar or concrete fill
Metal lath in joint under cores with bolts
Mortar fill in cores with bolts
DETAIL B-1
Solid units laid with full and leveled bed and full and squeezed head joints
Half-length solid units at bolt locations
Full leveled mortar bed on metal lath
Metal lath in joint under and mortar fill in cores with bolts
DETAIL B-2
Std. units modified to form bond beam units laid with full and squeezed head joints.
Mortar or concrete fill in all cavities in bond beam
No. 3 bar horizontal reinf.
Full leveled mortar bed on metal lath
Metal lath in joint under and mortar fill in cores with bolts
DETAIL C-1
3/16" or No. 3 gage wire joint reinf.
DETAIL C-2
a) LIGHT HAZARD
b) MODERATE HAZARD
c) SEVERE HAZARD

Figure 51 - Recommended Methods of Capping Concrete Masonry Foundations Supporting Wood Frame Construction in Termite Infested Areas.

tion of enclosed areas such as crawl spaces under houses and porches, and attics; proper site sanitation including removal of all waste wood products, stumps, roots and dead timber from the site; and provision for free inspection of all wood structural members in the substructure area.

Ordinarily, no special protective measures will be required in "no hazard" areas other than observance of good construction practices as outlined in the preceding paragraph. Concrete masonry foundations of conventional design and capped in accordance with the recommendations presented in the sections on the design of plain and reinforced concrete masonry foundations will be suitable for use in such areas.

Recommended alternate details for capping concrete masonry foundation walls supporting wood frame superstructures in "light," "moderate," and "severe" termite hazard areas are given in Fig. 51(a), (b), and (c) respectively.

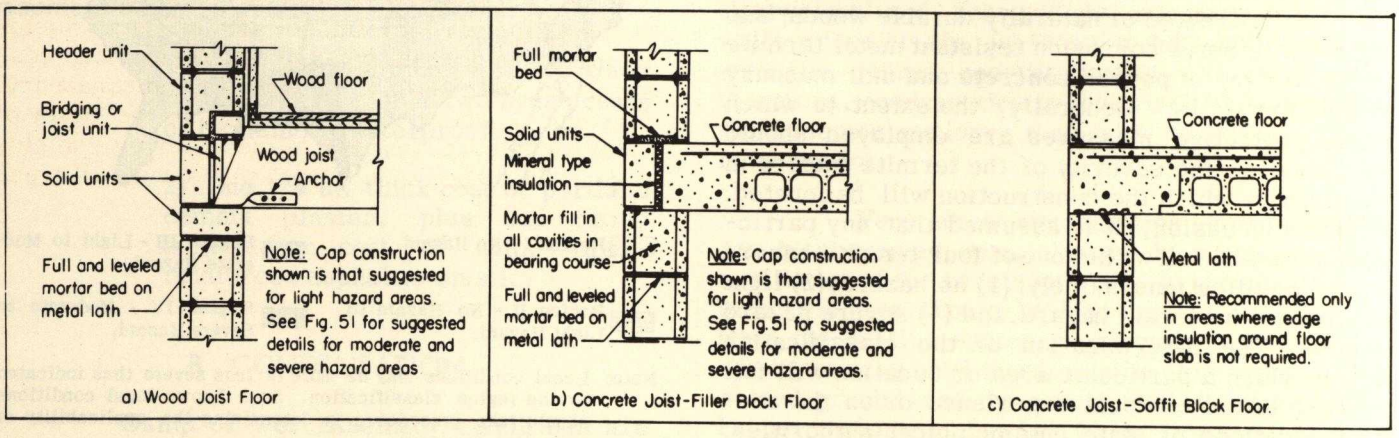

Header unit
Wood floor
Bridging or joist unit
Wood joist
Anchor
Solid units
Full and leveled mortar bed on metal lath
Note: Cap construction shown is that suggested for light hazard areas. See Fig. 51 for suggested details for moderate and severe hazard areas.
a) Wood Joist Floor
Full mortar bed
Concrete floor
Solid units
Mineral type insulation
Mortar fill in all cavities in bearing course
Full and leveled mortar bed on metal lath
Note: Cap construction shown is that suggested for light hazard areas. See Fig. 51 for suggested details for moderate and severe hazard areas.
b) Concrete Joist-Filler Block Floor
Concrete floor
Metal lath
Note: Recommended only in areas where edge insulation around floor slab is not required.
c) Concrete Joist-Soffit Block Floor.

Figure 52 - Recommended Methods of Capping Concrete Masonry Foundations Supporting Masonry Exterior Walls in Termite Infested Areas.

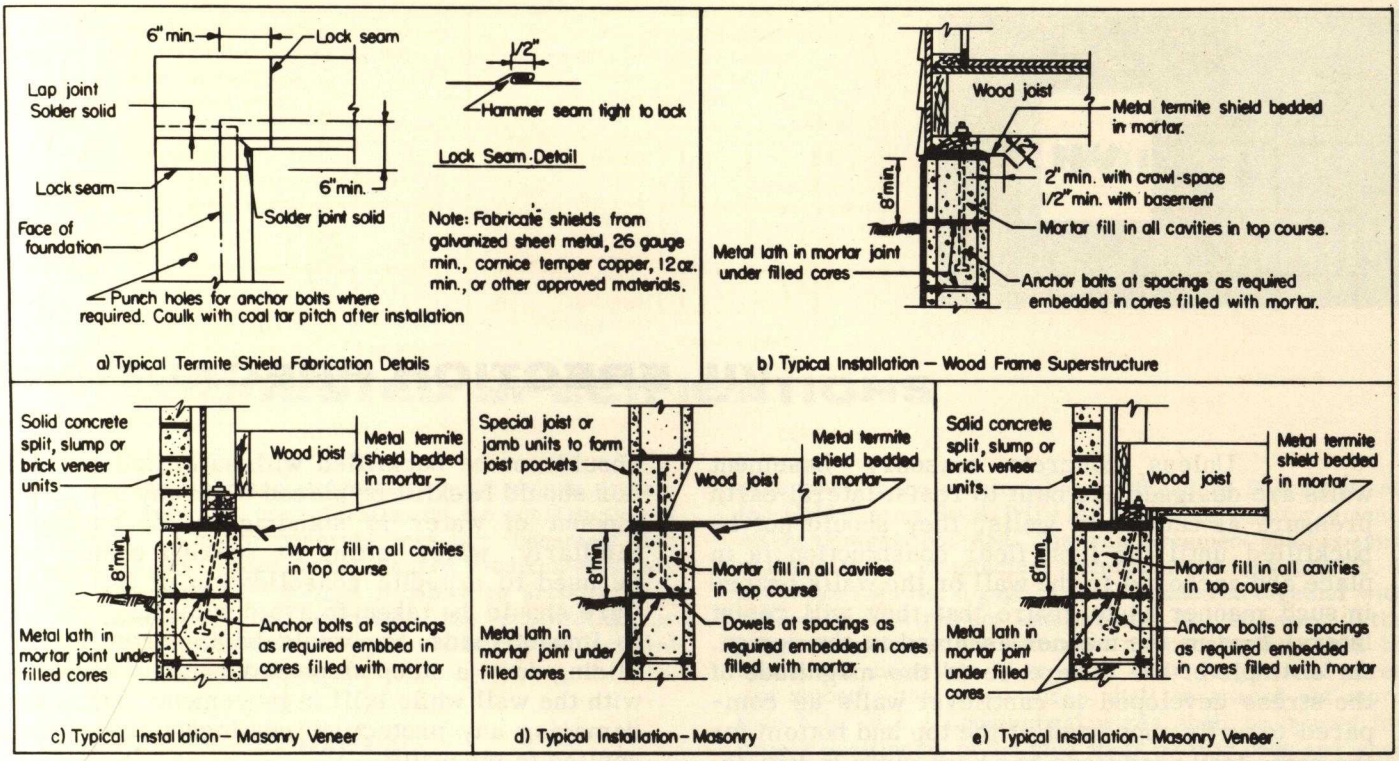

6" min.
Lock seam
Lap joint
Solder solid
Lock seam
6" min.
Face of foundation
Solder joint solid
Punch holes for anchor bolts where required. Caulk with coal tar pitch after installation
a) Typical Termite Shield Fabrication Details
1/2"
Hammer seam tight to lock
Lock Seam Detail
Note: Fabricate shields from galvanized sheet metal, 26 gauge min., cornice temper copper, 12 oz. min., or other approved materials.
Wood joist
Metal termite shield bedded in mortar.
8" min.
2" min. with crawl-space
1/2" min. with basement
Mortar fill in all cavities in top course.
Metal lath in mortar joint under filled cores
Anchor bolts at spacings as required embedded in cores filled with mortar.
b) Typical Installation — Wood Frame Superstructure
Solid concrete split, slump or brick veneer units
Wood joist
Metal termite shield bedded in mortar
8" min.
Mortar fill in all cavities in top course
Metal lath in mortar joint under filled cores
Anchor bolts at spacings as required embbed in cores filled with mortar
c) Typical Installation — Masonry Veneer
Special joist or jamb units to form joist pockets
Wood joist
Metal termite shield bedded in mortar
8" min.
Mortar fill in all cavities in top course
Dowels at spacings as required embedded in cores filled with mortar.
Metal lath in mortar joint under filled cores
d) Typical Installation — Masonry
Solid concrete split, slump or brick veneer units.
Wood joist
Metal termite shield bedded in mortar
8" min.
Mortar fill in all cavities in top course
Anchor bolts at spacings as required embedded in cores filled with mortar
Metal lath in mortar joint under filled cores
e) Typical Installation - Masonry Veneer.

Figure 53 - Termite Shield Details.

Similar details for walls supporting masonry superstructures are given in Fig. 52. Concrete masonry foundations capped in accordance with these recommended details and used in conjunction with proper site sanitation, provision for easy inspection of wood members in the substructure area, and other sound construction practices ordinarily will constitute adequate protection against attack by subterranean termites. Where attack by drywood and dampwood termites as well as by subterranean termites must be considered, where it is impossible to provide for free inspection of the substructure, or where local conditions dictate, additional protective measures may be justified. In such cases, soil treatment, chemically treated or naturally durable woods, or metal termite shields may be used in conjunction with concrete masonry foundations to provide the maximum possible protection against attack consistent with the importance of the construction.

When considering the use of concrete masonry foundations in areas of known termite infestation, it should be noted that some authorities require that such foundations be capped with 4 in. of poured concrete reinforced with wire mesh. It is believed that equally effective protection is obtained more practically with the recommended cap constructions shown in Figs. 51 and 52. Also, most authorities seem to agree that metal termite shields are effective barriers where they are properly designed, installed and maintained. Recommended details covering the installation of termite shields at the top of concrete masonry foundations are shown in Fig. 53. As illustrated in this figure, termite shields should be wide enough to extend over and project beyond the face of the foundation so as to bar the upward travel of termites on the surface of the wall as well as within cracks and other internal pathways. In this respect, the need for and function of metal shields is the same for all types of foundation wall construction, poured concrete as well as unit masonry. Therefore, where it is believed, on the basis of local experience, that use of metal termite shields is justified, they should be required with all types of foundations.

VII. ERECTION PRECAUTIONS

Unless concrete masonry basement walls are designed and built to resist lateral earth pressure as cantilever walls, they should not be backfilled until the first floor construction is in place and anchored to the wall or the walls braced in such manner as to insure that they will resist lateral load in the manner assumed in the design. An example of the difference in the magnitude of the stress developed in cantilever walls as compared to walls supported at the top and bottom for the same load conditions has been given in Fig. 15, page 22, and illustrates why premature backfilling should be avoided.

Similarly, unless basement walls are specifically designed to resist the higher earth pressures accompanying the application of heavy superimposed loads on the earth adjacent to the walls, heavy equipment, such as bulldozers or cranes, should not be operated over the backfill during the backfilling or other construction operations. Heavy superimposed loads on the ground adjacent to basement walls may increase earth pressures on the walls as illustrated in Fig. 2, page 5, and cause failure if the resultant total side pressure exceeds that which can be resisted by the walls.

Ordinarily, earth pressures assumed in the design of basement walls are selected on the assumption that the backfill material will be in a reasonably dry condition when placed. Since lateral earth pressures will increase as the moisture content of the earth is increased, approaching hydrostatic pressure at saturation, basement walls should not be backfilled with saturated materials nor should backfill be placed when any appreciable amount of water is standing in the excavation. Similarly, water jetting or soaking should never be used to expedite consolidation of the backfill. Care should be taken to avoid subjecting the walls to impact loads, as would be imparted by earth sliding down a steep slope and coming into contact with the wall while still in movement, and to avoid damaging any protective coatings or membranes applied to the walls.

The need for special bracing of partially completed construction should be considered whenever inclement weather halts the construction operations. Heavy rains have, in some instances, caused collapse of the side slopes of excavations or earth slides resulting in filling of the excavation adjacent to basement walls with earth and water to a point where the lateral pressures on the walls have been sufficient to cause failure. In areas where heavy rains are not unusual and would channel considerable surface water towards the foundation walls during construction, it is a practice of some contractors to omit an occasional block close to the bottom of the wall. In the event of flooding of the excavated area adjacent to the walls, these openings prevent the water level from building up to excessive heights on one side. The pressure on the inside and outside of the walls is thus equalized and the danger of wall failure due to excessive lateral pressures reduced. The block are installed in the wall after the lateral support for the top of the wall is in place and prior to backfilling.

VIII. SUGGESTED SPECIFICATIONS

The following suggested specifications pertain only to the selection of materials and the construction of plain and reinforced concrete masonry basement and foundation walls. They do not include requirements relative to the concrete footings required to support such walls, tile drains, other portions of the construction, or general requirements which would ordinarily be included in a complete specification.

A. MATERIALS

1.1 Concrete Masonry Units: Concrete masonry units shall be Grade A units conforming to the physical requirements of the following applicable standard specifications:

Hollow Load-Bearing Units; ASTM Designation: C90
Solid Load-Bearing Units; ASTM Designation: C145
Concrete Building Brick; ASTM Designation: C55

1.2 Mortar: Mortar used in the construction of concrete masonry basement and foundation walls shall be Type ___ conforming to the requirements of the following standard specification:

Mortar for Unit Masonry; ASTM Designation: C270

(Note: Insert the type or types of mortars which will be permitted in the foundation wall construction. Type M (A-1) mortar is recommended in reinforced concrete masonry walls. Type M (A-1) or the portland cement-lime Type S(A-2) are recommended in non-reinforced foundation walls.)

1.3 Reinforcement:

1.3.1 Steel bar reinforcement shall conform to the requirements of either of the following standard specifications:

Billet-Steel Bars for Concrete Reinforcement; ASTM Designation: A15
Rail-Steel Bars for Concrete Reinforcement; ASTM Designation: A16

No. 3 and larger bars shall be deformed with deformations conforming to:

Minimum Requirements for the Deformations of Deformed Steel Bars; ASTM Designation: A305

1.3.2 Horizontal joint reinforcement shall be of a prefabricated type and made with cold-drawn steel wire meeting the requirements of Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement; ASTM Designation: A82. The reinforcement shall consist of two parallel longitudinal members, not less than No. 9 nor more than No. 6 gage in size, connected together with welded cross wires not smaller than No. 12 gage. If the longitudinal wires are deformed the cross wires shall be spaced not more than 16 in. o.c. If the longitudinal wires are smooth, the cross wires shall be spaced not more than 6 in. o.c. Longitudinal wires shall be spaced for proper positioning in the face shell mortar beds and when intended for use in walls exposed to weather or moisture shall be of galvanized wire stock.

1.4 Concrete or Grout for Filling Reinforced Cores: Concrete or grout used to fill the reinforced cores or cavities in reinforced concrete masonry shall have an ultimate compressive strength at 28 days of at least 2500 psi. The grout or concrete shall consist of portland cement and aggregate complying with applicable ASTM specifications and shall be mixed to a suitable consistency for pouring without segregation of the constituents. Admixtures shall not be used unless approved as to type, quality and quantity. Maximum size of coarse aggregate in concrete shall not exceed 1/2 in. Where grout with no coarse aggregate is used, the sand shall have a fineness modulus of from 2.5 to 3.5.

(Note: Suggested proportions for concrete and grout corefill are given in the text of this booklet.)

B. WALL CONSTRUCTION

2.1 Workmanship - General:

2.1.1 All masonry walls shall be true, plumb and built to the thickness and bond or joint pattern indicated on the plans. Where no bond or joint pattern is indicated, units shall be laid in straight even courses using a running bond pattern with the units in the course above regularly breaking joints with the course below. All masonry shall be laid in accordance with accepted standards of good practice and workmanship in masonry construction. Special units shall be furnished and used where indicated on the plans and cutting of units shall be avoided insofar as possible.

2.1.2 The first course of masonry shall be laid on the footing in a full mortar bed. Mortar joints between units shall be 3/8 in. thick with full mortar coverage on vertical and horizontal face shells only. Vertical joints shall be shoved tight.

2.1.3 Mortar joints which are to be exposed or painted shall be struck off flush with the wall surface and, when the mortar has partially set, shall be firmly compacted with a (round) (or "V") pointing tool. Mortar joints in the face of walls to be plastered shall be struck off flush with the face of the wall.

2.1.4 The cross webs forming vertical cores to be filled shall be fully bedded in mortar to prevent grout leakage. The masonry shall be built to preserve the unobstructed continuity of the cores to be filled and such cores shall have a continuous unobstructed cell area of not less than 2 x 3 in. If the corefill is to be placed after the mortar has set and hardened, mortar protruding into the cores shall be removed or smoothed flush as the wall is laid up or broken loose by rodding and removed prior to the filling operation.

2.2 Materials:

2.2.1 Masonry units shall be clean, dry and free from ice and frost when laid in the wall. Masonry units shall not be dampened before or during laying unless approved by the architect.

2.2.2 Mortar not used within two and one-half hours after initial mixing or which has stiffened due to hydration past the point of initial set shall be discarded. Mortar which has stiffened due to evaporation shall be retempered to restore its workability by remixing and by addition of water as required.

2.2.3 Reinforcement shall be free of dirt, mud or heavy or loose rust when placed and shall consist of straight bars or wire except for bends at corners or where bends or hooks are detailed on the plans.

2.3 Placement of Reinforcement and Corefill:

2.3.1 Horizontal and vertical reinforcement shall be provided according to the details shown on the plans. Placement of grout or concrete fill shall not begin until all reinforcement in the cavities to be filled is in place and properly tied in position. Horizontal reinforcement in bond beams shall be placed as the wall is laid up. Vertical reinforcement may be set during the erection of the wall or after the wall has been laid up and shall be held in position at the top and bottom and at intervals not exceeding 160 bar diameters. Splices may be made only at such points and in such manner that the structural strength will not be reduced. Bars shall be lapped a minimum of 30 bar diameters at all splices unless otherwise indicated on plans. The minimum clear distance between the masonry units and vertical bars shall be 1/2 in.

2.3.2 All wall cavities containing reinforcement shall be completely filled with grout or concrete. The height of pour in filling the cells in masonry walls shall not exceed twelve times the sum of the unobstructed clear dimensions of the vertical cavity when filled with grout, nor more than ten times when filled with concrete, and in no case shall exceed 10 ft. Fill shall be rodded or vibrated during placement to insure complete filling. Where the height of the pour will exceed 4 ft., cleanouts shall be left at the base of all vertical cells to be filled. Such cleanouts shall be not closed nor filling started until after removal of mortar droppings, and setting and inspection of the vertical steel. The fill shall be placed in lifts not exceeding 4 ft. in height, rodded or vibrated during placement, and at least fifteen minutes allowed between successive lifts to permit settlement. When the time interval between lifts will exceed one hour, the lifts shall be stopped 1-1/2 in. below the top of a course so that the joint between lifts does not fall at a mortar joint. Wire screen, small mesh expanded metal lath or other approved material shall be placed in the mortar joint immediately under each bond beam course so as to cover and prevent filling of vertical cells not intended to be filled.

2.4 Bond and Anchorage:

2.4.1 Intersecting masonry walls and partitions shall be tied together with masonry bond or with approved metal ties in accordance with the details shown on the plans.

2.4.2 Anchor bolts, bearing plates, and other anchors or ties shall be built into the walls at all points and in the manner indicated on the plans.

2.4.3 Ends of floor joists or beams bearing on the masonry walls, and floor joists or beams adjacent to and extending parallel to masonry walls shall be securely fastened or anchored to the walls in accordance with the details shown on the plans.

2.5 Waterproofing:

2.5.1 The earth side of concrete masonry basement walls shall be parged with portland cement plaster applied in two coats, each coat 1/4 in. thick. Plaster shall consist of the mortar mix used in laying the units or a mixture of one volume portland cement, one-quarter volume hydrated lime and approximately three volumes mortar sand with the addition of enough water to produce a plastic mix. Wall surfaces to be plastered shall be clean and free of dirt, oil, or grease and moistened prior to application of the first coat. The first coat shall be roughened when partially set, permitted to harden for 24 hours, and moistened prior to application of the second coat. The second coat shall be kept damp for at least 48 hours. The cement plaster shall be thickened and rounded to form a cove at the junction of the outside wall face and footing.

(Note: Alternate methods of waterproofing as discussed in Section VI may be substituted for the above.)

2.6 Protection of Materials and Construction:

2.6.1 Concrete masonry units and cementitious materials shall be stored at the job site in a manner which will protect such materials from contact with the soil and weather.

2.6.2 Incomplete construction shall be adequately protected at all times and at such time as work is discontinued, the top of all walls shall be protected by a well secured waterproof cover.

2.6.3 Unless adequate provisions are made for heating and drying materials and for protecting the completed work, no masonry work shall be permitted when the ambient temperature is less than 32 F. or below 40 F. and falling. No masonry shall be laid on frozen materials.

2.6.4 Concrete masonry foundation and basement walls shall not be backfilled until the first floor is in place and anchored to the walls and the plaster surface or other waterproofing, where required, has cured sufficiently to resist damage. Heavy equipment shall not be operated over the backfill close to the basement walls. Backfill shall be carefully placed so as to avoid subjecting the walls to impact loads.

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   A 16-54T Specifications for Rail-Steel Bars for Concrete Reinforcement

   A 82-34 Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement

   A305-56T Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement

   C 33-56T Specifications for Concrete Aggregates

   C 39-56T Method of Test for Compressive Strength of Molded Concrete Cylinders

   C 55-55 Specifications for Concrete Building Brick

   C 90-52 Specifications for Hollow Load-Bearing Concrete Masonry Units

   C 91-56T Specifications for Masonry Cement

   C140-56 Methods of Sampling and Testing Concrete Masonry Units

   C145-52 Specifications for Solid Load-Bearing Concrete Masonry Units

   C150-56 Specifications for Portland Cement

   C175-56 Specifications for Air-Entraining Portland Cement

   C207-49 Specifications for Hydrated Lime for Masonry Purposes

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APPENDIX A

PROVISIONS OF THE 1954 EDITION OF THE AMERICAN STANDARD BUILDING CODE REQUIREMENTS FOR MASONRY RELATIVE TO THE DESIGN AND CONSTRUCTION OF PLAIN CONCRETE MASONRY FOUNDATION WALLS

SECTION 1. GENERAL

1.1. SCOPE. This standard covers requirements suitable for use in building codes, applying to the design and construction of masonry in building construction. It does not include requirements for reinforced concrete, reinforced gypsum concrete, or reinforced masonry, nor requirements concerning fire protection.

1.3. DIMENSIONS. Unless the word "actual" is used, the dimensional requirements for masonry given in this standard are nominal, and apply to such dimensions as minimum thicknesses and heights of walls. The measured dimensions of the masonry may be less than the required nominal dimensions by the thickness of one mortar joint, but not more than one-half inch, and by the amount of the dimensional tolerance permitted by specifications applying to units used in the construction.

1.5. TESTS.

1.5.1. WHEN REQUIRED. Whenever there is insufficient evidence that any material or construction conforms to these requirements, or there is insufficient evidence to substantiate claims for alternate materials or construction, the building official may require tests as proof of compliance to be made at the expense of the applicant by an approved agency.

1.5.2. METHODS. Tests of masonry materials shall be made in accordance with generally recognized standards. Strength tests of masonry constructions shall be made in accordance with the Tentative Methods of Conducting Strength Tests of Panels for Building Construction, ASTM E 72-51 T. Duly authenticated tests by a competent person or laboratory may be accepted by the building official in lieu of tests under his own supervision.

SECTION 2. MATERIALS

2.1. QUALITY. Materials used in masonry shall be of good quality conforming to generally accepted good practice. Except as may be otherwise provided herein, the standards and requirements set forth in Section 2.2, Standards of Quality, shall be deemed to represent generally accepted good practice in building construction. (See Section 1.5.2.)

2.2. STANDARDS OF QUALITY.

2.2.1.3. CONCRETE BRICK. American Standard Specifications for Concrete Building Brick (ASTM C 55-52; ASA A75.1-1953). Concrete brick subject to the action of weather or soil shall be of Grade A.

2.2.3. CONCRETE MASONRY UNITS

2.2.3.1. HOLLOW LOAD-BEARING UNITS. American Standard Specifications for Hollow Load-Bearing Concrete Masonry Units (ASTM C 90-52; ASA A79.1-1953), for hollow units in load-bearing masonry or subject to the action of weather or soil.

2.2.3.3. SOLID LOAD-BEARING UNITS. American Standard Specifications for Solid Load-Bearing Concrete Masonry Units (ASTM C 145-52; ASA A81.1-1953). Solid units subject to the action of weather or soil shall be Grade A.

2.2.9. CEMENTITIOUS MATERIALS.

Hydrated Lime for Masonry Purposes, ASTM C 207-49.
Hydraulic Hydrated Lime for Structural Purposes, ASTM C 141-42.
Quicklime for Structural Purposes, ASTM C 5-26.
Masonry Cement, ASTM C 91-52T.
Natural Cement, ASTM C 10-52T.
Portland Cement, ASTM C 150-52.
Air-Entraining Portland Cement, ASTM C 175-51 T.
Portland Blast Furnace Cement, ASTM C 205-51 T.

2.2.10. AGGREGATE FOR MORTAR. Standard Specifications for Aggregate for Masonry Mortar, ASTM C 144-52 T.

2.2.12. WATER. Water shall be clean, and free from injurious amounts of oils, acids, alkalis, organic materials, or other deleterious substances.

SECTION 3. MORTAR

3.1. QUALITY. Mortar shall conform to the requirements of ASTM Tentative Specifications for Mortar for Unit Masonry, ASTM Designation C 270-52T.

3.4. TYPES OF MORTAR PERMITTED. Masonry shall be laid in mortar of the types specified in Table 1.

TABLE 1.

Kind of masonry	Types of mortar permitted
Foundations:	
Footings	A–1 or A–2
Walls of solid units	A–1, A–2 or B
Walls of hollow units	A–1 or A–2
Hollow walls	A–1 or A–2

SECTION 4. ALLOWABLE STRESSES

4.1. GENERAL. In determining the stresses in the masonry, the effects of all loads and conditions of loading and the influence of all forces affecting the design and strength of the several parts shall be taken into account. Stresses shall be calculated on actual rather than nominal dimensions. Masonry construction shall be so designed and constructed that the allowable stresses prescribed herein are not exceeded.

4.2. ALLOWABLE STRESSES. Except as permitted in other sections of this standard, the compressive stresses in masonry shall not exceed the values given in Table 2.

TABLE 2.
ALLOWABLE COMPRESSIVE STRESSES IN UNIT MASONRY

Construction; grade of unit	Allowable compressive stresses gross cross-sectional area (except as noted) *				
	Type A-1 mortar	Type A-2 mortar	Type B mortar	Type C mortar	Type D mortar
Solid masonry of solid concrete masonry units:					
Grade A	175	160	140		
Grade B	125	115	100		
Masonry of hollow units	85	75			
Hollow walls (cavity or masonry bonded).†					
Solid units:					
Grade A or 2,5000, psi plus	140	130			
Grade B or 1,500 to 2,500, psi.	100	90			
Hollow units	70	60			
Piers of hollow units, cellular spaces filled	105	95			

* Values given for Types C and D mortar with solid units and for Type B mortar with hollow and filled cell hollow units have been omitted as they are not applicable to foundation walls.

† On gross cross-sectional area of wall minus area of cavity between wythes (leaves). The allowable compressive stresses for cavity walls are based upon the assumption that the floor loads bear upon but 1 of the 2 wythes. When hollow walls are loaded concentrically, the allowable stresses may be increased 25 percent.

4.2.1. HIGHER STRESSES. Higher stresses than herein specified may be used, but only if it is clearly established to the satisfaction of the building official, by test, or other approved evidence, that material of a higher grade or a superior workmanship than is generally provided in accepted practice will be employed under approved inspection. The use of higher stresses, however, shall not be allowed until a statement giving the reasons for such permission together with the facts and circumstances on which it is based, has been placed on file and made a part of the official record of the permit.

4.2.2. COMPOSITE WALLS. In composite walls or other structural members composed of different kinds or grades of units or mortars, the maximum stress shall not exceed the allowable stress for the weakest of the combinations of units and mortars of which the member is composed.

SECTION 6. THICKNESS OF MASONRY

6.4. FOUNDATION WALLS.

6.4.1. THICKNESS. Foundation walls shall be of sufficient strength and thickness to resist lateral pressures from adjacent earth and to support their vertical loads without exceeding the allowable stresses. Foundation walls or their footings shall extend below the level of frost action and shall be not less in thickness than the walls immediately above them except as provided in Section 6.4.2.5. Foundation walls shall be at least 12 inches thick except as noted in Section 6.4.2, Exceptions.

6.4.2. EXCEPTIONS.

6.4.2.1. REINFORCED MASONRY. Foundation walls of masonry units supported laterally at vertical intervals not exceeding 12 feet may be 8 inches thick if reinforced by means of vertical 3/8-inch diameter deformed reinforcing bars (or their equivalent) spaced not more than 12 inches apart and not less than 3-1/2 inches from the pressure side of the wall; or 1/2-inch diameter deformed reinforcing bars (or their equivalent) spaced not more than 24 inches apart and not less than 5 inches from the pressure side of the wall. Reinforcing bars shall extend from footing to top of foundation wall. Bars shall be lapped at least 30 diameters at splices. The space between each bar and the adjacent enclosing masonry surfaces shall be solidly filled with Type A-1 grout or mortar.

6.4.2.2. ALLOWABLE DEPTH OF 8-INCH FOUNDATION WALLS. Solid foundation walls of solid masonry units or of coursed stone that do not extend more than 5 feet below the adjacent finished ground level may be 8 inches in thickness; cavity walls and walls of hollow units that do not extend more than 4 feet below the adjacent finished ground level, may be 10 inches and 8 inches, respectively, in thickness. Those depths may be increased to a maximum of 7 feet with the approval of the building official when he is satisfied that soil conditions warrant such increase. The combined height of an 8-inch foundation wall and the wall supported shall not exceed the height permitted for 8-inch walls.

6.4.2.5. FOUNDATION WALLS SUPPORTING BRICK VENEER OR CAVITY WALLS. Foundation walls of 8-inch thickness and conforming to the provisions of this section may be used as foundations for dwellings with walls of brick veneer on frame walls, or with 10-inch cavity walls; provided that the dwelling is not more than 1-1/2 stories in height and the total height of the wall, including the gable, is not more than 20 feet. Foundation walls of 8-inch thickness supporting brick veneer or cavity walls shall be corbelled with solid units to provide a bearing the full thickness of the wall above unless adequate bearing is provided by a concrete floor slab. The total projection shall not exceed 2 inches with individual corbels projecting not more than 1/3 the depth (height) of the unit. The top corbel course shall be not higher than the bottom of floor joists and shall be a full header course of headers at least 6 inches in length.

SECTION 7. BOND

7.1. WALLS OF SOLID MASONRY UNITS. Solid masonry bearing and nonbearing walls shall be bonded in accordance with Section 7.1.1, Bonding with Bonders or Section 7.1.2, Bonding with Metal Ties.

7.1.1. BONDING WITH BONDERS (HEADERS). The facing and backing shall be bonded so that not less than 4 percent of the wall surface of each face is composed of bonders (headers) extending not less than 4 inches into the backing. The distance between adjacent full-length headers shall not exceed 24 inches either vertically or horizontally. In walls in which a single bonder does not extend through the wall, bonders from the opposite sides shall overlap at least 4 inches, or bonders from opposite sides shall be covered with another bonder course overlapping the bonder below at least 4 inches.

7.1.2. BONDING WITH METAL TIES. The facing and backing shall be bonded with corrosion-resistant metal ties conforming to the requirements of Section 7.6 for cavity walls. There shall be one metal tie for not more than each 4-1/2 square feet of wall area. Ties in alternate courses shall be staggered, the maximum vertical distance between ties shall not exceed 18 inches, and the maximum horizontal distance shall not exceed 36 inches. Walls so bonded shall conform to the allowable stress, lateral support, thickness (excluding cavity), height and mortar requirements for cavity walls.

7.2. MASONRY WALLS OF HOLLOW UNITS. Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches by lapping at least 4 inches over the unit below, or by lapping at vertical intervals not exceeding 17 inches with units which are at least 50 percent greater in thickness than the units below; or by bonding with corrosion-resistant metal ties conforming to the requirements of Section 7.6.1. There shall be one metal tie for not more than each 4-1/2 square feet of wall area. Ties in alternate courses shall be staggered, the maximum vertical distance between ties shall not exceed 18 inches, and the maximum horizontal distance shall not exceed 36 inches. Walls bonded with metal ties shall conform to the requirements for allowable stress, lateral support, thickness (excluding cavity), height and mortar for cavity walls. Hollow masonry units shall have full mortar coverage of the ends and edges of the face shells in both the horizontal and vertical joints.

7.6. HOLLOW WALLS (CAVITY AND MASONRY BONDED).

7.6.1. CAVITY WALLS. The facing and backing of cavity walls shall be bonded with 3/16-inch diameter steel rods or metal ties of equivalent stiffness embedded in the horizontal joints. There shall be one metal tie for not more than each 4-1/2 square feet of wall area. Ties in alternate courses shall be staggered as indicated in Section 7.1.2, the maximum vertical distance between ties shall not exceed 18 inches, and the maximum horizontal distance shall not exceed 36 inches. Rods bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical; in other walls the ends of ties shall be bent to 90-degree angles to provide hooks not less than 2 inches long. Additional bonding ties shall be provided at all openings, spaced not more than 3 feet apart around the perimeter and within 12 inches of the opening. Ties shall be of corrosion-resistant metal, or shall be coated with a corrosion-resistant metal, or other approved protective coating.

7.6.2. MASONRY-BONDED HOLLOW WALLS. Masonry bonded hollow walls shall be bonded as required in Section 7.1.1, Bonding with Bonders (Headers).

7.7. BONDING OF INTERSECTING WALLS.

7.7.1. WHERE BONDED. Masonry walls shall be securely anchored or bonded at points where they intersect and where they abut or adjoin the frame of a skeleton frame building. (See Section 11.1 for anchoring to floors.)

7.7.2. BONDING BEARING WALLS. When two bearing walls meet or intersect and the courses are built up together, the intersections shall be bonded by laying in a true bond at least 50 percent of the units at the intersection.

7.7.3. WALLS CARRIED UP SEPARATELY. When the courses of meeting or intersecting bearing walls are carried up separately, the intersecting walls shall be regularly toothed or blocked with 8-inch maximum offsets and the joints provided with metal anchors having a minimum section of 1/4 inch by 1-1/2 inches with ends bent up at least 2 inches, or with cross pins to form anchorage. Such anchors shall be at least 2 feet long and the maximum spacing shall be 4 feet.

SECTION 11. MISCELLANEOUS MASONRY REQUIREMENTS

11.1. ANCHORING OF WALLS.

11.1.1. MEETING OR INTERSECTING WALLS. Masonry walls that meet or intersect floors or columns shall be securely bonded or anchored if these are depended upon for lateral support of the walls. (See Section 7.7, Bonding of Intersecting Walls.)

11.1.2. FASTENING JOISTS OR BEAMS. The ends of floor joists or beams bearing on masonry walls shall be securely fastened to the walls in an approved manner.

11.1.3. SPACING. When lateral support for walls is to be provided by anchorage to floor or roof joists which are parallel to the walls, the anchors shall be spaced at intervals not exceeding 6 feet and shall engage not less than three joists; these joists shall be bridged solidly at the anchors.

11.1.4. ANCHORING STRUCTURAL MEMBERS. Structural members including roof members, framing into or supported by walls or piers shall be adequately anchored.

11.2. CHASES AND RECESSES.

11.2.1. LIMITATIONS. Except as provided in Section 11.2.1.1, chases and recesses in masonry walls shall not be deeper than 1/3 the wall thickness nor longer than 4 feet horizontally or in horizontal projection, and shall have at least 8 inches of masonry in back of the chases and recesses and between adjacent chases or recesses and the jambs of openings. Chases and recesses shall not be cut in walls of hollow masonry units or in hollow walls, but when permitted may be built in. There shall be no chases or recesses within the required area or a pier. The aggregate area of recesses and chases in any wall shall not exceed 1/4 of the whole area of the face of the wall in any story.

11.2.1.1. EXCEPTIONS FOR 8-INCH WALLS. In residence buildings not over 2 stories in height, vertical chases not more than 4 inches deep and not more than 4 square feet of wall area may be built in 8-inch walls, except that recesses below windows may extend from floor to sill and be the width of the opening above. Masonry directly above chases or recesses wider than 12 inches shall be supported on lintels.

11.2.2. RECESSES FOR STAIRWAYS OR ELEVATORS. Recesses for stairways or elevators may be left in walls, but in no case shall the walls at such points be reduced to less than 12 inches unless reinforced by additional piers, or by columns or girders of steel, reinforced masonry, or concrete, securely anchored to the walls on each side of such recesses. Recesses for alcoves and similar purposes shall have not less than 8 inches of material at the back. Such recesses shall be not more than 8 feet in width and shall be arched over or spanned with lintels.

11.3. LINTELS AND ARCHES.

11.3.1. MATERIAL. The masonry above openings shall be supported by arches or lintels of metal or reinforced masonry which shall bear on the wall at each end for not less than 4 inches. Stone or other nonreinforced masonry lintels shall not be used unless supplemented on the inside of the wall with iron or steel lintels or with suitable masonry arches or reinforced masonry lintels carrying the masonry backing.

11.3.2. STIFFNESS. Steel or reinforced masonry lintels shall be of sufficient stiffness to carry the superimposed load without deflection of more than 1/360 of the clear span.

11.3.3. DESIGN. Masonry arches shall be designed in accordance with good engineering practice.

11.4. BEAMS AND JOIST SUPPORTS.

11.4.1. BEAMS. Beams, girders, or other concentrated loads supported by a wall or pier, shall have bearing at least 3 inches in actual length upon solid masonry not less than 4 inches in depth or on a metal bearing plate of adequate design and dimensions, or other provision shall be made to distribute safely the loads on the wall or pier.

11.4.2. JOISTS. Joists shall have bearing at least 3 inches in actual length upon solid masonry at least 2-1/4 inches in depth, or other provision shall be made to distribute safely the loads on the wall or pier.

11.4.3. SUPPORT ON WOOD. Unless specifically approved by the building official, no masonry shall be supported on wood girders or other form of wood construction.

11.5. CORBELLING.

11.5.1. MAXIMUM PROJECTION. Except for chimneys, the maximum corbelled horizontal projection beyond the face of the wall shall be not more than 1/2 the wall thickness and the maximum projection of one unit shall neither exceed 1/2 the depth of the unit nor 1/3 its width at right angles to the face which is offset.

11.5.2. CORBELLING CHIMNEYS. No chimney shall be corbelled from a wall more than 6 inches nor shall a chimney be corbelled from a wall which is less than 12 inches in thickness, unless it projects equally on each side of the wall, provided that in the second story of two-story dwellings, corbelling of chimneys on the exterior of the enclosing walls may equal the wall thickness. In every case the corbelling shall not exceed 1 inch projection for each course of brick projected.

11.9. PRECAUTIONS DURING ERECTION.

11.9.1. BRACING TO RESIST LATERAL LOADS. Masonry walls in locations where they may be exposed to high winds during erection shall not be built higher than 10 times their thickness unless adequately braced or until provision is made for the prompt installation of permanent bracing at the floor or roof level immediately above the story under construction. Back fill shall not be placed against foundation walls until they have been braced to withstand the horizontal pressure.

11.9.3. PROTECTION AGAINST FREEZING. Masonry shall be protected against freezing for at least 48 hours after being laid. Unless adequate precautions against freezing are taken, no masonry shall be built when the temperature is below 32 F. on a rising temperature, or below 40 F. on a falling temperature, at the point where the work is in progress. No frozen materials shall be built upon.

APPENDIX B

TYPICAL REINFORCED CONCRETE MASONARY BASEMENT WALL CALCULATIONS

I. NOMENCLATURE

A = net cross-sectional area of contact between block and mortar in plain masonry walls or in unreinforced portions of unreinforced walls. See Fig. 38 (c), page 45.

A_a = average net cross-sectional area of reinforced walls considered effective in direct compression. See Fig. 38 (a), page 45.

A_s = effective cross-sectional area of steel reinforcement.

b = breadth of wall section under consideration with rectangular sections and width of compression flange of T-beam sections in stress computations.

b' = assumed effective width in shear stress computations. See Fig. 38 (c), page 45.

C = coefficient in deflection formula.

C_m = coefficient of resistance of T-beam with respect to masonry.

C_s = coefficient of resistance of T-beam with respect to steel.

d = depth from compression face of flexural member to centroid of tensile reinforcement.

D = deflection.

e = eccentricity; distance from line of action of resultant of compressive load to the centroid of the wall.

e_m = maximum compressive unit strain in extreme fiber of masonry.

e_s = maximum tensile unit strain in tensile reinforcement.

E_m = instantaneous modulus of elasticity of masonry in compression.

E'_m = sustained modulus of elasticity of masonry in compression.

E_s = modulus of elasticity of steel in tension.

f_a = computed axial compressive stress in masonry.

f_m = allowable stress in masonry in axial compression, flexural compression, or bearing, and computed flexural compressive stress in combined stress formula.

f'_m = 28 day compressive strength of masonry.

f_s = computed and allowable tensile stress in steel reinforcement.

f'_s = allowable flexural tensile stress in steel reinforcement in members sustaining axial compressive loads.

F_a = allowable axial compressive stress in masonry in combined stress formula.

F_m = allowable flexural compressive stress in masonry in combined stress formula.

j = ratio of distance between centroid of compression and centroid of tension to the depth "d" in flexural members.

k = ratio of distance from extreme fiber in compression to the neutral axis to the depth "d" in flexural members; coefficient dependent on type of live load or structure in calculating earthquake forces.

L = clear span.

M = moment acting on flexural member.

M_e = allowable earth pressure moment.

M_{rd} = resisting moment limited by deflection.

M_{rm} = resisting moment limited by compressive stress in masonry.

M_{rs} = resisting moment limited by tensile stress in tensile reinforcement.

n = ratio of modulus of elasticity of steel to that of masonry.

Σo = summation of perimeters of bars serving as tensile reinforcement.

p = ratio of tensile reinforcement in wall flexural members = $\frac{A_s}{bd}$.

P = axial compressive load.

t' = minimum face shell thickness of concrete masonry units and thickness of compression flange of T-beam sections of reinforced walls.

u = allowable and computed bond stress between steel reinforcement and masonry.

v = allowable shearing stress in masonry with properly designed shear reinforcement.

v_m = allowable and computed shearing stress in reinforced masonry without shear reinforcement.

v'_m = allowable shear stress in unreinforced masonry.

V_e = allowable earth pressure shear.

V_m = total shear acting on section or total shearing resistance of wall section.

V_u = calculated shearing resistance of reinforced walls with respect to bond stresses.

V_v = calculated shearing resistance of reinforced section of reinforced walls.

V'_v = calculated shearing resistance of unreinforced portions of reinforced walls.

II. TYPICAL CALCULATIONS.

Example 1. A concrete masonry basement wall is to support an axial compressive load of 2500 lb. per lin. ft. and to extend 8 ft. below grade in a soil with an equivalent-fluid weight of 25 lb. per cu. ft. (Type A soil). The wall will have a clear height of 10 ft. from the top of the basement slab to the bottom of the first floor. Determine the recommended wall thickness and vertical reinforcement required if all of the earth pressure load is to be carried in the vertical span.

Solution: Any of the following wall designs may be selected depending upon the wall thickness and reinforcement which will provide the most suitable and economical construction:

A. Reinforced Wall Designs - See Table 14, page 39, or Fig. 33, page 37.

10 in. Wall: No. 5 bars at 24 in. o. c.
No. 6 bars at 32 in. o. c.
No. 7 bars at 40 in. o. c.

The bars should be positioned 7 in. from the compression face of the wall and the walls reinforced horizontally with not less than 0.077 sq. in. of reinforcement per foot of wall height located in bond beams or as joint reinforcement. In this case, the recommended reinforcement is governed by the minimum reinforcement requirements and is in excess of that indicated by the wall stress calculations.

B. Partially Reinforced Wall Designs - See Table 15, page 40 and 41, or Figs. 35 and 36, page 42.

10 in. Wall: No. 5 bars at 32 in. o. c.
No. 6 bars at 40 in. o. c.
No. 7 bars at 48 in. o. c.

The bars should be positioned 7 in. from the compression face of the wall and the wall reinforced horizontally with at least 2- No. 5 bars located in a bond beam at the top of the wall or equivalent joint reinforcement.

12 in. Wall: No. 5 bars at 48 in. o. c.
No. 6 bars at 64 in. o. c.
No. 7 bars at 72 in. o. c.
No. 8 bars at 80 in. o. c.

The bars should be positioned 8-3/4 in. from the compression face of the wall and the wall reinforced horizontally with at least 2- No. 5 bars located in a bond beam at the top of the wall or equivalent joint reinforcement.

Example 2. Determine the recommended wall thickness and vertical reinforcement required for the same conditions given in Example 1 except that the axial compressive load is increased to 5000 lb. per lin. ft. and the soil assumed to have an equivalent-fluid weight of 40 lb. per cu. ft. (Type B soil).

Solution: Any of the following wall designs may be selected depending upon the wall thickness and reinforcement which will provide the most suitable and economical construction:

A. Reinforced Wall Designs - See Table 14, page 39, or Fig. 33, page 37.

10 in. Wall: No. 6 bars at 24 in. o. c.
No. 7 bars at 32 in. o. c.

The bars should be positioned 7 in. from the compression face of the wall and the wall reinforced horizontally with not less than 0.077 sq. in. of reinforcement per foot of wall height located in bond beams or as joint reinforcement.

B. Partially Reinforced Wall Designs - See Table 15, page 41, or Fig. 36, page 42.

12 in. Wall: No. 5 bars at 32 in. o. c.
No. 6 bars at 40 in. o. c.

The bars should be positioned 8-3/4 in. from the compression face of the wall and the wall reinforced horizontally with not less than 2- No. 5 bars located in a bond beam at the top of the wall or equivalent joint reinforcement.

Example 3: A concrete masonry basement wall, 12 in. thick and reinforced vertically with No. 7 bars spaced 32 in. o.c., is to support an axial compressive load of 5000 lb. per lin. ft. The wall height from top of the basement floor slab to the bottom of the first floor will be 12 ft. and the wall will be reinforced horizontally to meet the recommended minimum reinforcement requirements for reinforced walls. Determine the allowable wall depth below grade for both Type A and Type B soils assuming:

1) the earth pressure load will be carried entirely in the vertical span;

2) the wall will be simply supported at the top and bottom;

3) an ultimate masonry compressive strength of 1600 psi;

4) the wall units will have minimum face shell and cross web thicknesses of 1-1/2 and 1-1/4 in., vertical bars will be positioned 8-3/4 in. from the compression face of the wall, and the width of the reinforced cores plus the crosswebs forming the cores will be not less than 5-1/4 in.

Solution:

1) Calculated Axial Compressive Stress:

$$f_a = \frac{P}{A_a}$$

$$A_a = [5.25(11.625) + 2(1.5)(32 - 5.25)]\ \frac{12}{32}$$

$$A_a = 53 \text{ sq. in./ft.}$$

$$f_a = \frac{5000}{53} = \underline{94 \text{ psi}}$$

2) Allowable Flexural Compressive Stress:

$$\frac{f_a}{F_a} + \frac{f_m}{F_m} = 1.0$$

$$f_m = (1.0 - \frac{f_a}{F_a})F_m$$

$$f_m = (1.0 - \frac{94}{320})\ 528 = \underline{373 \text{ psi}}$$

3) Allowable Tensile Steel Stress:

$$f'_s = f_s + nf_a$$

$$f'_s = 20000 + 18.75(94) = \underline{21{,}770 \text{ psi}}$$

(Note: The working stress in the steel is taken higher than 20,000 psi to allow for the reduction in the actual stress in the steel due to the axial compressive load.)

4) Resisting Moment of Wall Section: Considering the reinforced sections in the wall as comparable to T-beams with the effective width of the compression flange limited so its overhanging width on either side of the web does not exceed eight times the minimum face shell thickness. See Fig. 38 (b), page 45.

a) Section properties:

$$b = 5.25 + (2)(8)(1.5) = 29.25 \text{ in.}$$

$$d = 8.75 \text{ in.}; \quad bd = 256 \text{ sq. in.}; \quad bd^2 = 2240 \text{ cu. in.}$$

$$p = \frac{A_s}{bd} = \frac{0.225}{256} = 0.00234; \quad \frac{t'}{d} = \frac{1.5}{8.75} = 0.171$$

$$pn = 0.00234\,(18.75) = 0.044$$

From "Concrete Reinforcing Steel Institute Design Handbook";

$$k = 0.27; \qquad j = 0.928$$

b) Resisting moment limited by steel stress:

$$M_{rs} = C_s \frac{f's}{n} bd^2$$

From "CRSI Design Handbook"; $C_s = 0.04$

$$M_{rs} = (0.04)\frac{21770}{18.75}(2240) = 103{,}800 \text{ in. lb.}$$

$$M_{rs} = 103800\left(\frac{12}{32}\right) = \underline{38{,}900 \text{ in. lb./ft.}}$$

c) Resisting moment limited by masonry stress:

$$M_{rm} = C_m f_m bd^2$$

From "CRSI Design Handbook"; $C_m = 0.109$

$$M_{rm} = 0.109(373)(2240) = 91{,}000 \text{ in. lb.}$$

$$M_{rm} = 91000\left(\frac{12}{32}\right) = \underline{34{,}100 \text{ in. lb./ft.}}$$

d) Resisting moment limited by deflection:

$$M_{rd}{}^{*} = \frac{24250\, bd^3}{\left(\frac{2}{C_m} + \frac{1}{C_s}\right) L}$$

$$M_{rd} = \frac{24250(19570)}{(43.4)(144)} = 76{,}000 \text{ in. lb.}$$

$$= 76000\left(\frac{12}{32}\right) = \underline{28{,}500 \text{ in. lb./ft.}}$$

* Derivation of equation for allowable moment limited by deflection

Taking formula suggested by Maney,

$$D = \frac{CL^2}{d}(e_m + e_s)$$

Substituting:

$$e_m = \frac{f_m}{E'_m} \; ; \; e_s = \frac{f_s}{E_s} \; ; \; E'_m = \frac{1}{2} E_m$$

$$f_m = \frac{M_{rd}}{C_m bd^2} \; ; \; f_s = \frac{nM_{rd}}{C_s bd^2}$$

$$D = \frac{CL^2}{d}\left[\frac{\frac{M_{rd}}{C_m bd^2}}{\frac{1}{2}E_m} + \frac{\frac{nM_{rd}}{C_s bd^2}}{nE_m}\right]$$

Simplifying and transposing:

$$M_{rd} = \frac{DE_m bd^3}{CL^2\left(\frac{2}{C_m} + \frac{1}{C_s}\right)}$$

Substituting: $D = \frac{L}{600}$; $E_m = 1{,}600{,}000$ psi; $C = 0.11$

$$M_{rd} = \frac{24250\ bd^3}{\left(\frac{2}{C_m} + \frac{1}{C_s}\right) L}$$

5) Shearing Resistance of Wall Section:

a) Shearing resistance of reinforced section:

$$V_v = v_m b' jd$$

$$b' = 5.25 + 5(1.25) = 11.5 \text{ in.}$$

$$V_v = 32(11.5)(0.928)(8.75) = 2{,}990 \text{ lb.}$$

$$V_v = 2990\left(\frac{12}{32}\right) = \underline{1{,}120 \text{ lb./ft.}}$$

b) Shearing resistance of plain masonry between reinforced sections:

$$V'_v = v'_m A$$

$$A = 2\,(1.5)\,(32\text{-}29.25) = 8.25 \text{ sq. in.}$$

$$V'_v = 12\,(8.25)\left(\frac{12}{32}\right) = \underline{37 \text{ lb./ft.}}$$

c) Total shearing resistance of combined reinforced and plain masonry sections:

$$V = 1120 + 37 = \underline{1{,}157 \text{ lb./ft.}}$$

6) Shearing Resistance of Wall Section as Limited by Allowable Bond Stress:

$$V_u = u\Sigma ojd$$

$$V_u = 160(2.75)\,(0.928)(8.75) = 3{,}580 \text{ lb.}$$

$$V_u = 3580\ \left(\frac{12}{32}\right) = \underline{1{,}340 \text{ lb./ft.}}$$

7) Design Allowable Earth Pressure Moment and Shear:

Resisting moment of wall as limited by deflection:

$$M_{rd} = 28{,}500 \text{ in. lb./ft.}$$

Applied moment due to axial load acting on deflected wall:

$$D = L/600 = 144/600 = 0.24 \text{ in.}$$

$$M = Pe = 5000\ (0.24) = 1{,}200 \text{ in. lb./ft.}$$

Allowable earth pressure moment:

$$M_e = 28{,}500 - 1{,}200 = \underline{27{,}300 \text{ in. lb./ft.}}$$

Allowable earth pressure shear:

$V_e = \underline{1{,}157 \text{ lb./ft.}}$ limited by shearing resistance of the combined reinforced and plain masonry sections of the wall

8) Allowable Wall Depth Below Grade:

a) From Fig. 8, page 10: With Type A soil having an equivalent-fluid weight of 25 lb. per cu. ft. for a wall height of 12 ft. and a depth below grade of about 10.9 ft.:

$M_e = 27{,}300$ in. lb./ft.

$V_e = 1{,}050$ lb./ft.

* Wall may extend 10.9 ft. below grade in Type A soils.

b) From Fig. 9, page 11: With Type B soil having an equivalent-fluid weight of 40 lb. per cu. ft. for a wall height of 12 ft. and a depth below grade of about 8.6 ft.:

$M_e = 27{,}300$ in. lb./ft.

$V_e = 1{,}125$ lb./ft.

* Wall may extend 8.6 ft. below grade in Type B soils.

* These values are approximately equal to those shown in Table 14, page 39, and Fig. 34, page 37.

Example 4. A concrete masonry basement wall, 12 in. thick and reinforced vertically with No. 6 bars spaced 40 in. o.c., is to support an axial compressive load of 5000 lb. per lin. ft. The wall height from the top of the basement floor slab to the bottom of the first floor will be 10 ft. Determine the allowable wall depth below grade for both Type A and Type B soils assuming:

1) the wall will be considered to be partially reinforced;

2) the earth pressure load will be carried entirely in the vertical span;

3) an ultimate masonry compressive strength of 1600 psi;

4) unit dimensions and effective depth of reinforcement as in Example 3.

Solution:

1) Calculated Axial Compressive Stress on Net Section:

$$f_a = \frac{P}{A_a}$$

$$A_a = [5.25(11.625) + 2(1.5)(40-5.25)](\frac{12}{40})$$

$$A_a = 49.5 \text{ sq. in./ft.}$$

$$f_a = \frac{5000}{49.5} = \underline{101 \text{ psi}}$$

2) Allowable Flexural Compressive Stress:

$$f_m = (1.0 - \frac{f_a}{F_a}) F_m$$

$$f_m = (1.0 - \frac{101}{320}) 528 = \underline{361 \text{ psi } *}$$

* Although the allowable axial compressive load on partially reinforced walls is limited by the allowable axial compressive stress permitted on plain masonry, the allowable axial stress which would be permitted if the wall were fully reinforced should be used in determining the allowable flexural compressive stress.

3) Allowable Tensile Steel Stress:

$$f'_s = f_s + nf_a$$

$$f'_s = 20000 + 18.75(101) = \underline{21,890 \text{ psi}}$$

4) Resisting Moment of Wall Section:

a) Section properties: (See Example 3.)

$$b = 29.25 \text{ in.}; \; d = 8.75 \text{ in.}; \; \frac{t'}{d} = 0.171$$

$$bd = 256 \text{ sq. in.}; \; bd^2 = 2240 \text{ cu. in.};$$

$$p = \frac{A_s}{bd} = \frac{0.136}{256} = 0.00172$$

$$pn = 0.00172(18.75) = 0.0322$$

From "CRSI Design Handbook";

$$k = 0.23; \quad j = 0.932$$

b) Resisting moment limited by steel stress:

$$M_{rs} = C_s \frac{f'_s}{n} bd^2$$

From "CRSI Design Handbook"; $C_s = 0.03$

$$M_{rs} = (0.03) \frac{21890}{18.75} (2240) \frac{12}{40} = \underline{23,700 \text{ in. lb./ft.}}$$

c) Resisting moment limited by masonry stress:

$$M_{rm} = C_m f_m bd^2$$

From "CRSI Design Handbook"; $C_m = 0.10$

$$M_{rm} = 0.10(361)(2240)\frac{12}{40} = \underline{24,300 \text{ in. lb./ft.}}$$

d) Resisting moment limited by deflection: See Example 3 for derivation of deflection formula.

$$M_{rd} = \frac{24250 \, bd^3}{(\frac{2}{C_m} + \frac{1}{C_s}) L}$$

$$M_{rd} = \frac{24250(19570)}{(53.3)(120)} (\frac{12}{40}) = \underline{22,300 \text{ in. lb./ft.}}$$

5) Shearing Resistance of Wall Section:

a) Shearing resistance of reinforced section:

$$V_v = v_m b' jd$$

$$b' = 5.25 + 5(1.25) = 11.5 \text{ in.}$$

$$V_v = 32(11.5)(0.932)(8.75)\left(\frac{12}{40}\right) = \underline{900 \text{ lb./ft.}}$$

b) Shearing resistance of plain masonry between reinforced sections:

$$V'_v = v'_m A$$

$$A = 2(1.5)(40 - 29.25) = 32.2 \text{ sq. in.}$$

$$V_v = 12(32.2)\left(\frac{12}{40}\right) = \underline{116 \text{ lb./ft.}}$$

c) Total shearing resistance of combined reinforced and plain masonry sections:

$$V = 900 + 116 = \underline{1{,}016 \text{ lb./ft.}}$$

6) Shearing Resistance of Wall Section as Limited by Allowable Bond Stress:

$$V_u = u\Sigma ojd$$

$$V_u = 160(2.36)(0.932)(8.75)\left(\frac{12}{40}\right) = \underline{921 \text{ lb./ft.}}$$

7) Design Allowable Earth Pressure Moment and Shear:

Resisting moment as limited by deflection:

$$M_{rd} = 22{,}300 \text{ in. lb./ft.}$$

Applied moment due to axial load acting on deflected wall:

$$M = Pe = 5000(0.20) = 1{,}000 \text{ in. lb./ft.}$$

Allowable earth pressure moment:

$$M_e = 22{,}300 - 1{,}000 = \underline{21{,}300 \text{ in. lb./ft.}}$$

Allowable earth pressure shear:

$$V_e = \underline{921 \text{ lb./ft.}} \text{ limited by bond stress}$$

8) Allowable Depth Below Grade:

a) From Fig. 8, page 10: With Type A soil having an equivalent-fluid weight of 25 lb. per cu. ft. and for a wall height and depth below grade of 10 ft.:

$$M_e = 19{,}200 \text{ in. lb./ft.}$$

$$V_e = 833 \text{ lb./ft.}$$

* Wall may extend entirely below grade in Type A soils.

b) From Fig. 9, page 11: With Type B soil having an equivalent-fluid weight of 40 lb. per cu. ft. and for a wall height of 10 ft. and a depth below grade of about 7.9 ft.:

$$M_e = 18{,}900 \text{ in. lb./ft.}$$

$$V_e = 920 \text{ lb./ft.}$$

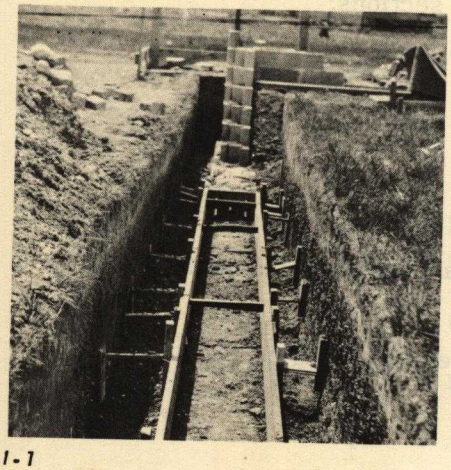
* Wall may extend 7.9 ft. below grade in Type B soils.

* These values are approximately equal to those shown in Table 15, page 41, and Fig. 36, page 42.

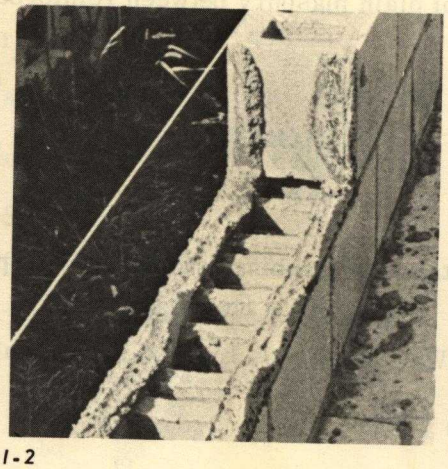
APPENDIX C

ILLUSTRATIONS OF TYPICAL CONCRETE MASONRY FOUNDATION WALL CONSTRUCTION

I. Plain Concrete Masonry Construction



I-1



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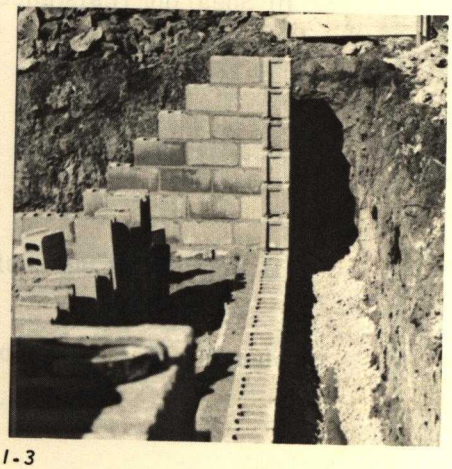

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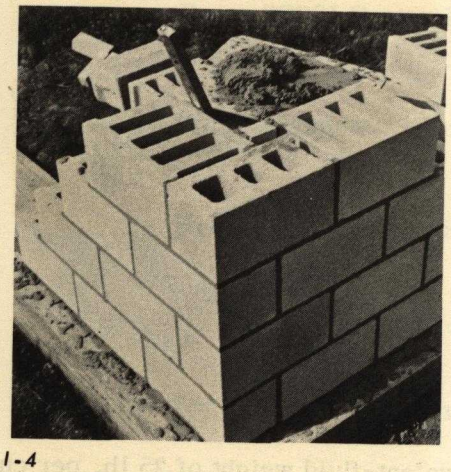


I-4

Fig. I-1. Concrete masonry foundations should be supported on a poured concrete footing bearing on firm soil well below the frost line and reinforced where necessary.

Fig. I-2. The first block course is set on the footing in a full mortar bed. Remaining courses generally are laid with face shell mortar bedding only.

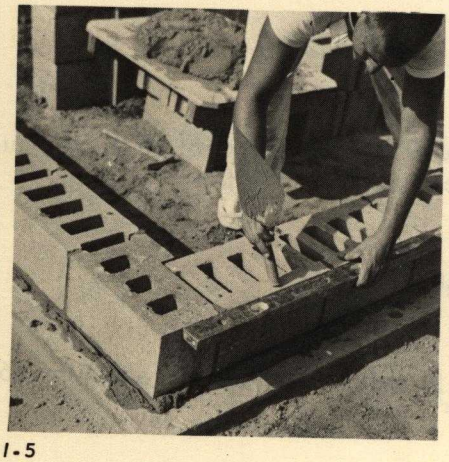
Fig. I-3. After setting of the first course, corners are built up 4 to 6 courses high and stretcher units then laid between the corners. A tile drain around the perimeter of footings for basement walls is recommended in most areas.

Fig. I-4. Special corner construction is required with wall thicknesses in excess of 8 in. An 8 in. unit and a concrete brick often are used in corner construction with 10 and 12 in. walls.

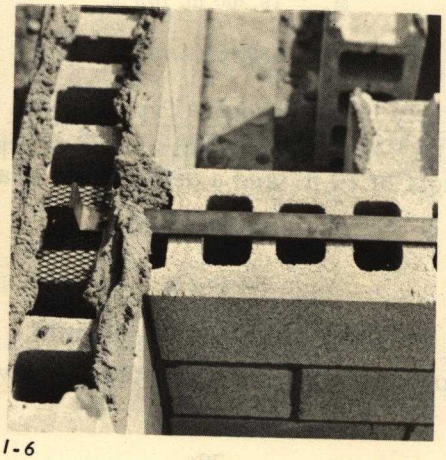
Fig. I-5. In some areas, special L-corner units are available for 10 in. and 12 in. wall corners.

Fig. I-6. Bearing walls should be tied together at intersections with metal anchors embedded in the mortar joints and with bent ends projecting into filled cores in each wall.

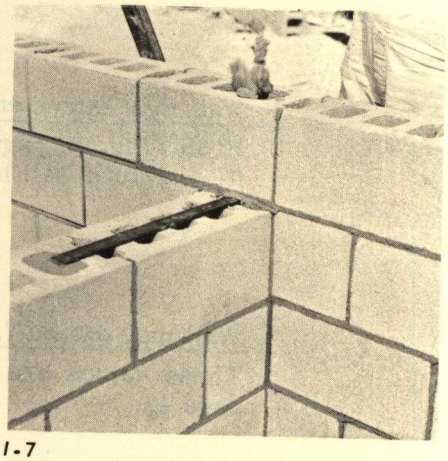
Fig. I-7. Cores containing bent ends of metal ties are filled with concrete or mortar to insure solid anchorage.



I-5

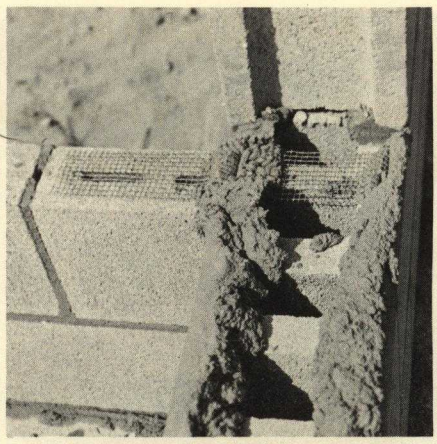


I-6



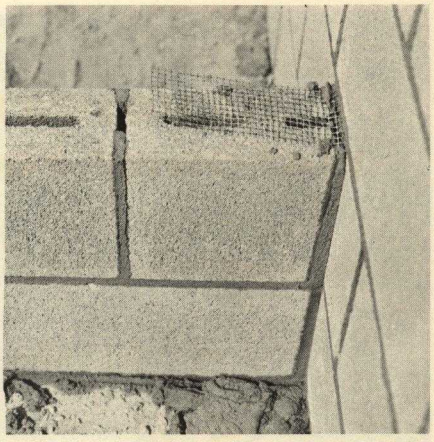
I-7

Fig. I-8. Non-bearing partitions should be tied to intersecting bearing or non-bearing walls. Strips of 1/4 in. mesh galvanized hardware cloth embedded in alternate joints may be used.



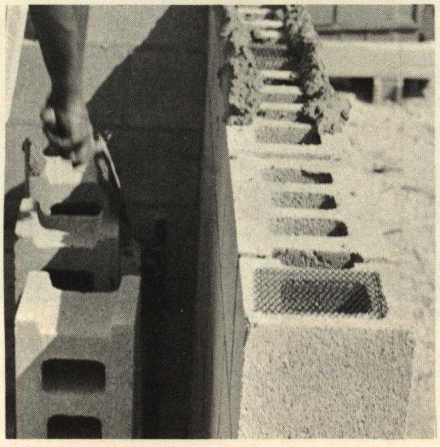
I-8

Fig. I-9. When intersecting walls are laid up separately, the metal ties are built into the first wall and later tied into the second, as in the case of this non-bearing partition.



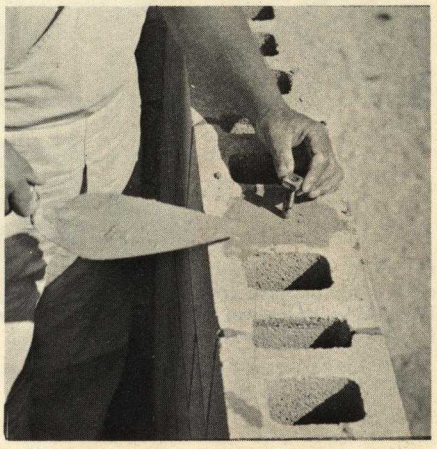
I-9

Fig. I-10. Strips of metal lath or wire screen set in the mortar joint under cores which will be filled with concrete or mortar prevents filling of cores under this joint.



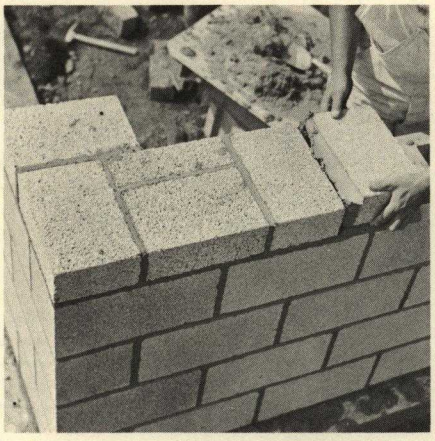
I-10

Fig. I-11. Anchor bolts are placed in cores of the top two block courses and these cores then filled with concrete or mortar.



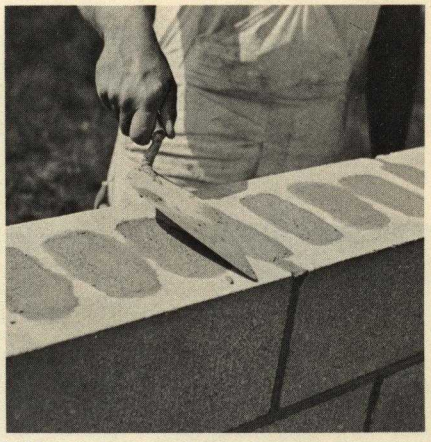
I-11

Fig. I-12. Hollow unit foundation walls must be capped with 4 in. of solid masonry or equivalent construction used in the bearing course.



I-12

Fig. I-13. Filling of all cores of hollow units in the bearing course is generally accepted as equivalent to 4 in. of solid masonry.

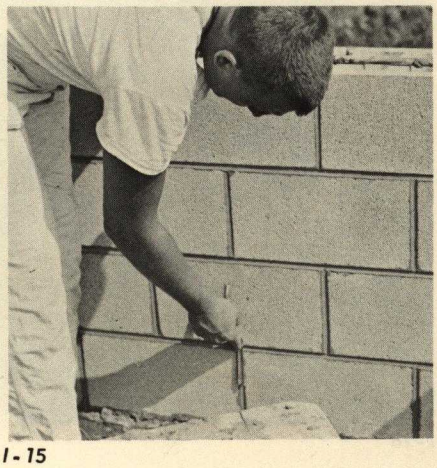


I-13

Fig. I-14. Solid top units are available and used in many areas to cap foundation walls and are considered equivalent to 4 in. of solid masonry.

I-14

Fig. I-15. Mortar joints should be firmly compacted with a rounded or V-shaped tool if the wall will not be plastered.

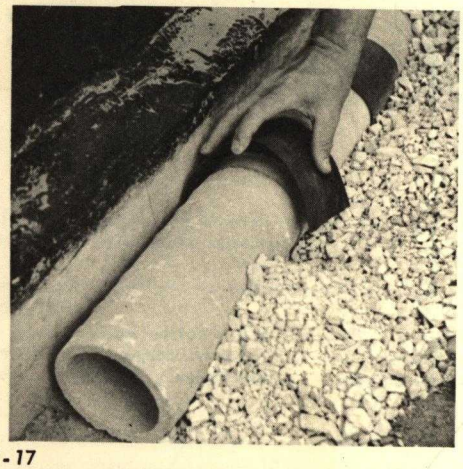


I-15

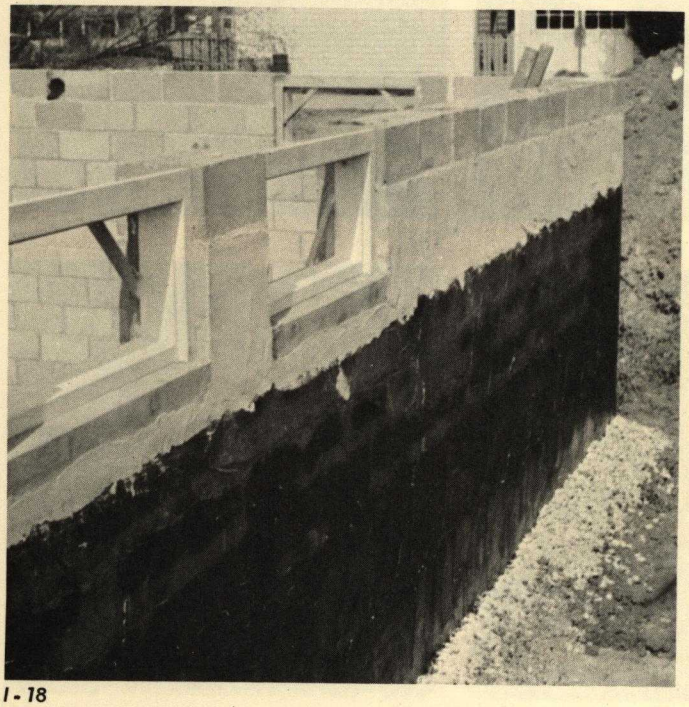
I-16

Fig. I-16. Two 1/4 in. thick coats of portland cement plaster applied to the exterior wall face will satisfactorily water-proof most concrete masonry basement walls. The plaster should be extended down over the footing to form a smooth cove and a tile drain installed around the perimeter of the footing.

Fig. I-17. When installing a tile drain, it is good practice to cover the open joints between tile with building felt and to cover the drain with at least 12 in. of coarse gravel or crushed stone to prevent dirt from entering and blocking the tile line.



I-17



I-18

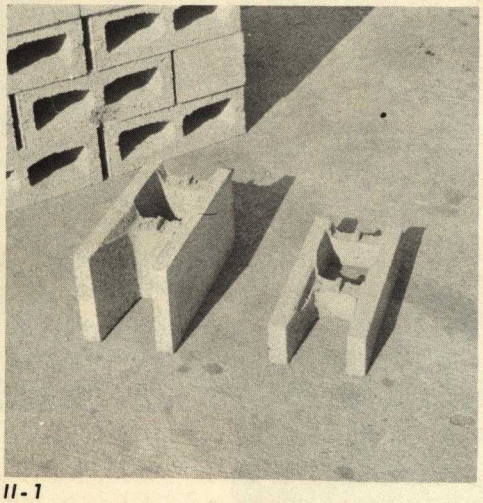
Fig. I-18. One or two coats of bituminous material sometimes is used in addition to or in lieu of portland cement plaster to waterproof basement walls. Both are used where a high degree of protection against moisture penetration is desired.

I-19

Fig. I-19. Interior of a concrete masonry basement illustrating conventional running bond wall construction.

I-20

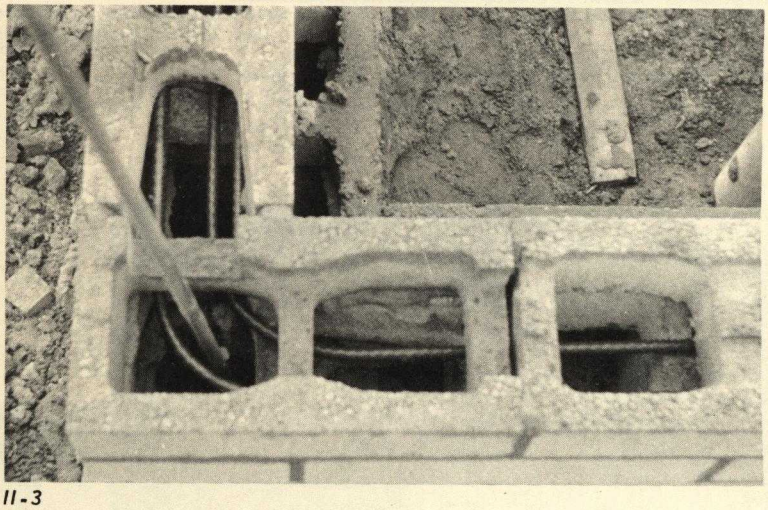
Fig. I-20. A concrete masonry basement in which standard 4 in. and 8 in. high units were used and laid in a coursed ashlar pattern to enhance the interior appearance.



II-1

II-2

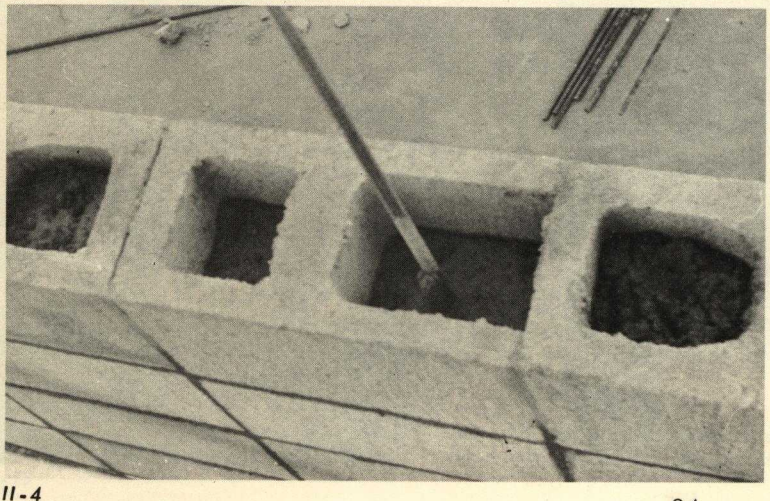
Fig. II-1. Special open-end units designed specifically for use in reinforced wall construction. They may be obtained both with and without cut-down webs or grooves, as shown, to permit placement of horizontal reinforcement. Open-end units are not available in all areas or from all producers.



II-3

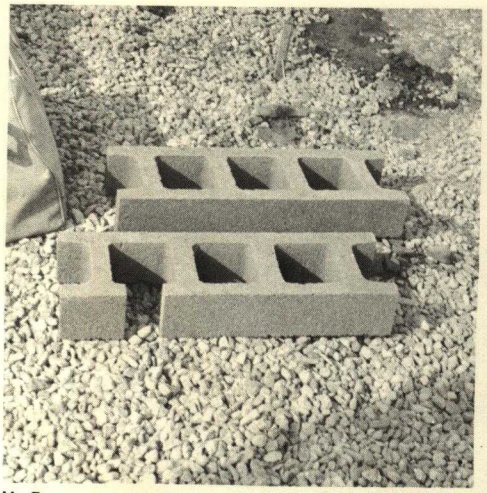
Fig. II-1. The first course, in reinforced walls, is laid on the footing in a full mortar bed as in plain masonry construction. In this case the vertical steel was cast in position in the poured footing, eliminating the need for dowels. Note how the open-end unit fits around the vertical steel.

Fig. II-3. Horizontal reinforcement is placed in bond beam courses as the wall construction progresses and lapped at corners or other points to insure continuity.

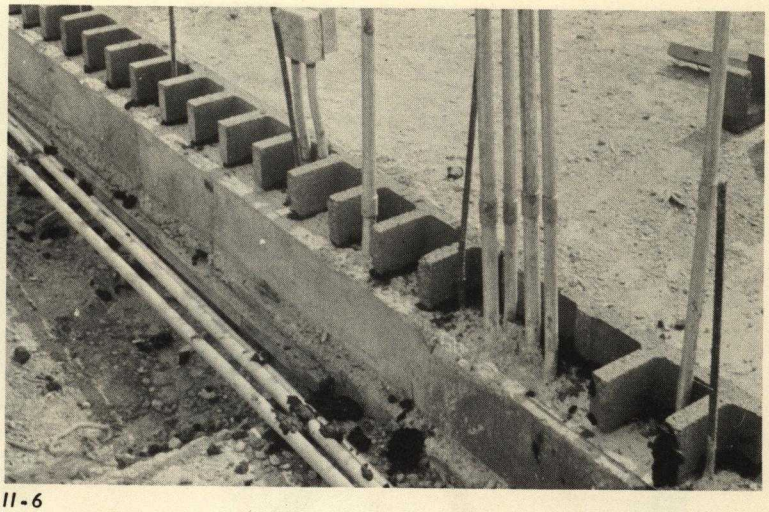


II-4

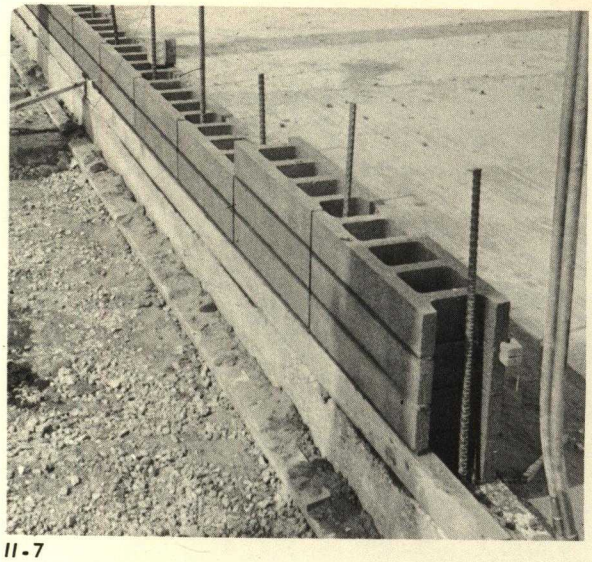
Fig. II-4. The grout or concrete fill may be placed simultaneously with the wall construction, at specified vertical intervals, or upon reaching each bond beam level. In this case, the fill concrete was placed in 16 in. lifts, all cores filled and each pour stopped below the top of the units in the top course.



II-5



II-6



II-7

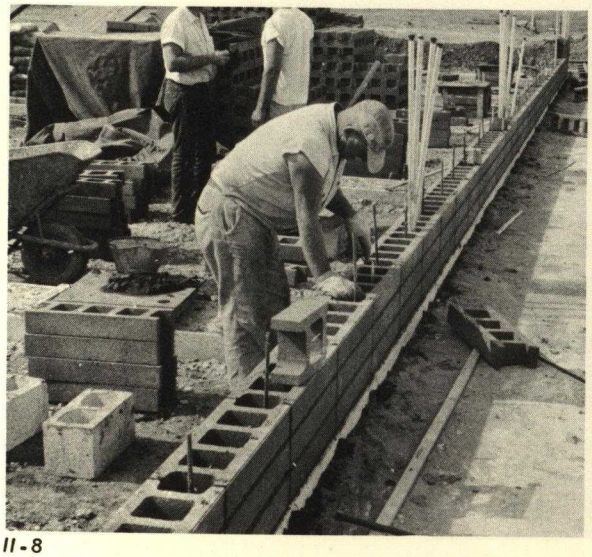
Fig. II-5. Concrete masonry units are made in various sizes and shapes, most of which are suitable for reinforced work. The 8 x 4 x 24 in. units shown in this and the other pictures on this page are not available in all areas, however, the construction techniques illustrated apply to other standard closed-end unit designs as well. A portion of the face shell of the unit in the foreground was removed to provide an opening for an electrical outlet.

Fig. II-6. First course of a wall in which the vertical steel and concrete embedment will be placed after the wall has been laid up. One face shell of the units for the first course has been removed to provide for inspection or clean-out openings.

Fig. II-7. A board may be used to form over the clean-out openings and temporarily support the upper courses. Note the dowels cast in the footing so as to project up into the wall. With the bond pattern and the spacing of reinforcement employed in this case, threading of the units over the dowels has been avoided.

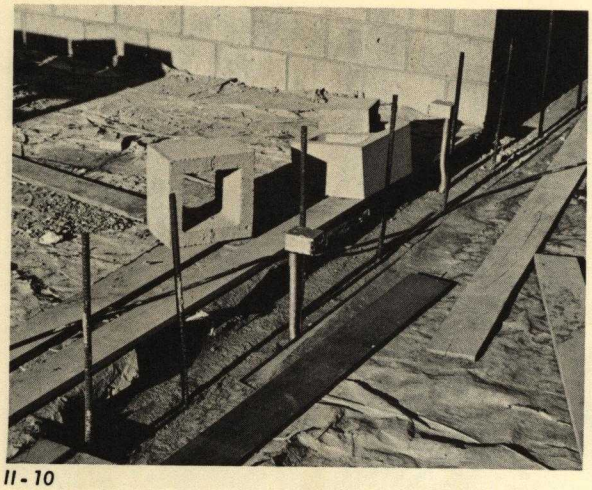
Fig. II-8. Mason removing mortar protrusions within the cores with a trowel. The walls will be continued up to story height, the vertical steel set and the cores then filled from the top.

Fig. II-9. A view of the clean-out course of the completed wall. After the pour was completed, the form board over the clean-outs was removed and the face shells, which had been removed to form the clean-outs, replaced.



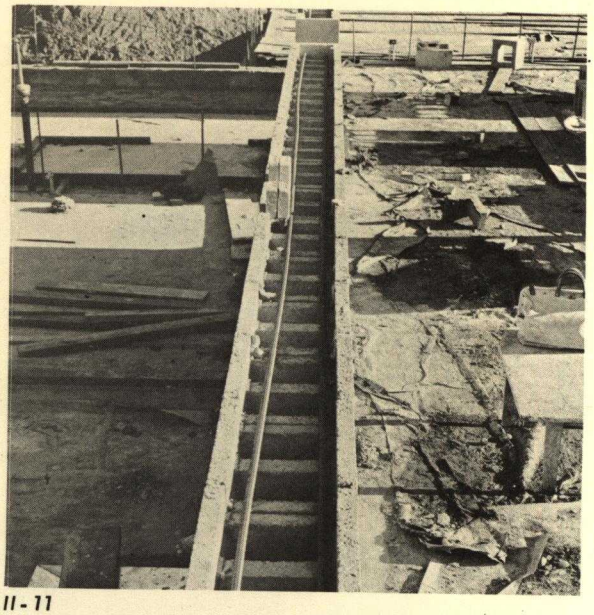
II-8

II-9



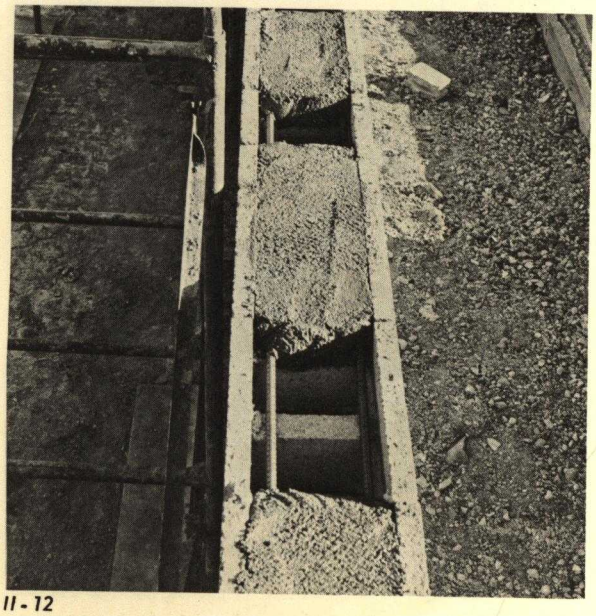
II - 10

Fig. II-10. Footing for a reinforced concrete masonry wall. Note the dowels cast in and projecting from the footing to anchor the wall at the base. The vertical steel will be placed and the cores filled from the top of the wall.



II - 11

Fig. II-11. Horizontal steel is placed in the bond beam courses as the construction progresses. In this case, standard two-core units with cut-down webs have been used to form the bond beam.



II - 12

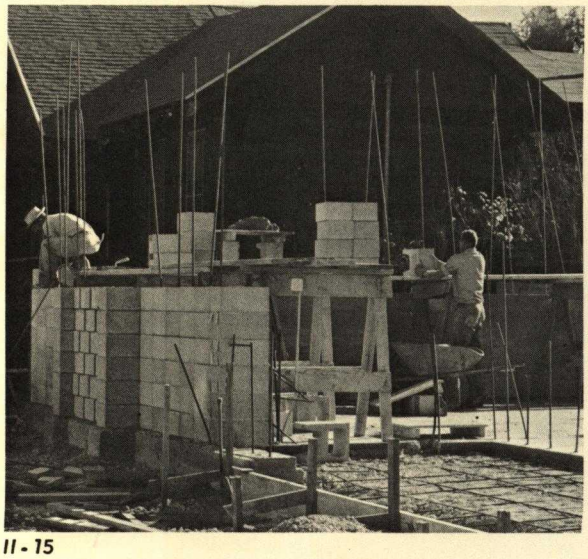
Fig. II-12. In some cases, to facilitate filling of a bond beam several block courses below the pour level, the portions of the bond beam between the vertical reinforced cells are poured as shown in this view before setting the next course of units. The vertical steel will be placed and the pour completed when the masonry has been laid to pour level.

Fig. II-13. Usual practice is to pour the vertical cells and the bond beams simultaneously, as in this view. The reinforcement in the bond beam shown in this view will be set before the pour is completed.

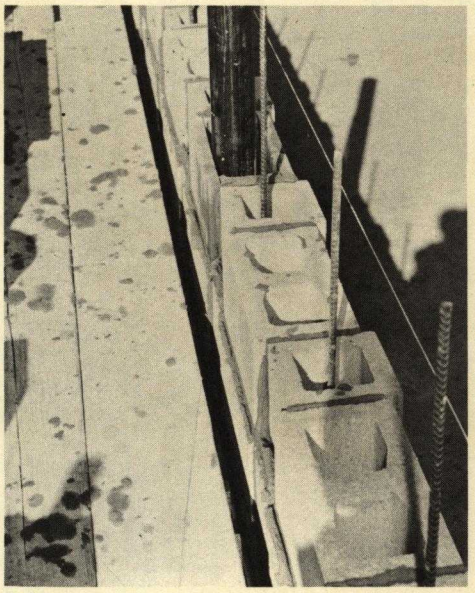
Fig. II-14. View of a two-story reinforced wall with construction completed to the second floor level. Note that the reinforcing is projected to tie the second story to the first story construction.

II - 14

II - 13

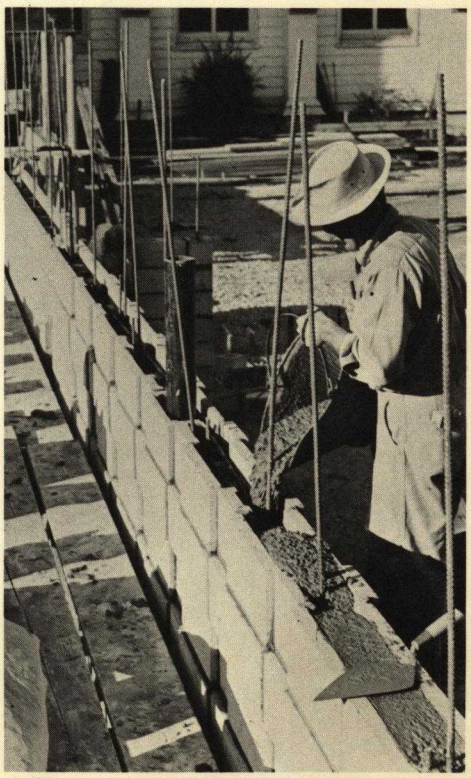


II - 15



II-17

II-16



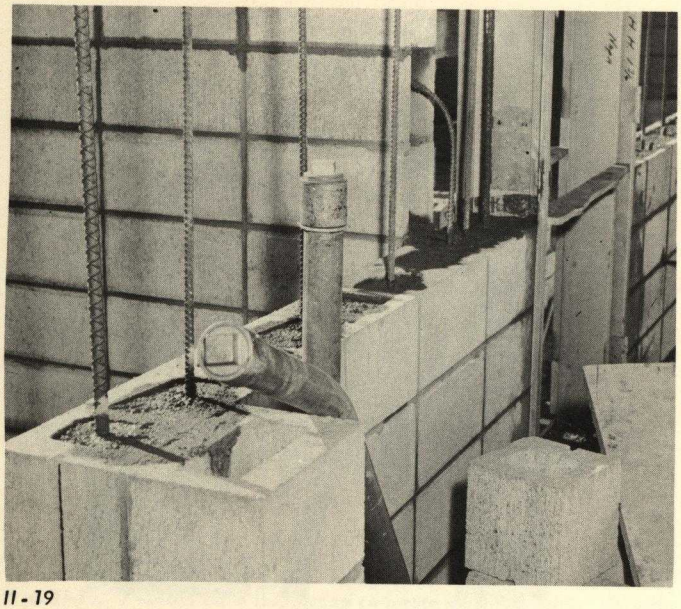
II - 18

Fig. II-15. View during construction of a reinforced wall where the vertical steel has been set in position before the units are laid. Open-end units are being used to eliminate threading of units over the vertical steel.

Fig. II-16. Another view illustrating open-end units as used in reinforced wall construction.

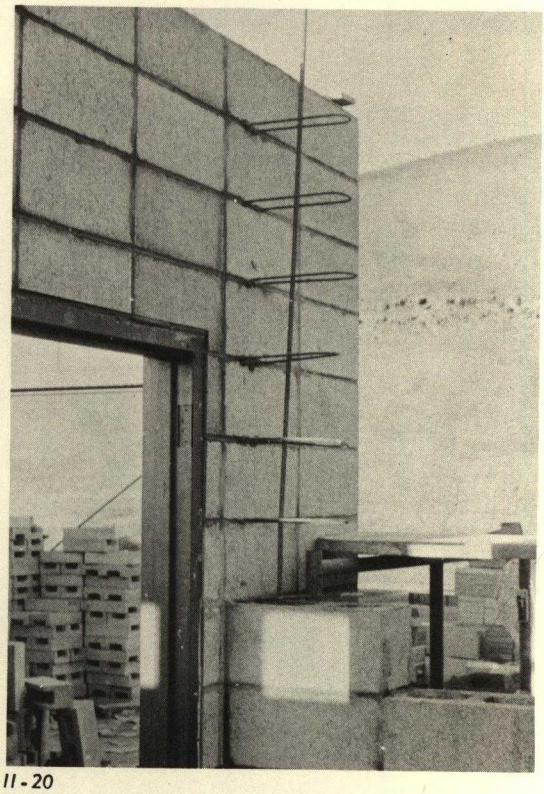
Fig. II-17. In both plain and reinforced construction, some units may be recessed to achieve pleasing decorative effects where desired. The use of recessed half-length units in conjunction with full length units is illustrated in this view.

Fig. II-18. Placing concrete in reinforced vertical cores and bond beam course. The use of coal buckets to fill the cells in reinforced walls as shown here is generally confined to small projects or pours. On large projects, cubic yard buckets, equipped with chutes and handled with lift trucks, are frequently used in the pouring operation.



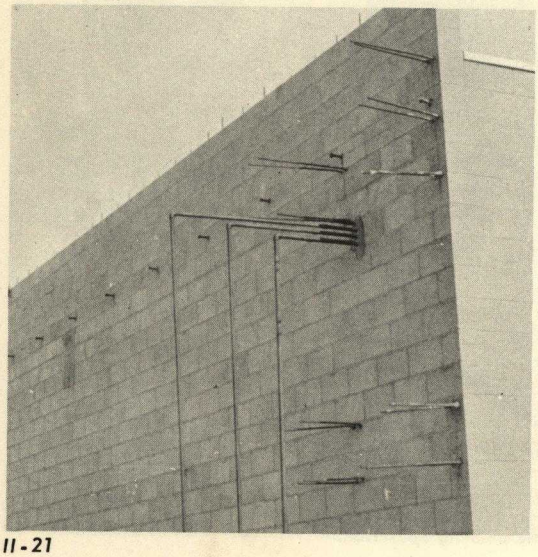
II-19

Fig. II-19. Reinforced walls may be bonded at intersections with metal ties embedded in reinforced cells in the two/walls. In this view, the tie is embedded in a bond beam in one wall and in a vertical core in the other. On this job, all cores in walls laid in stacked bond were filled, whether containing reinforcement or not.



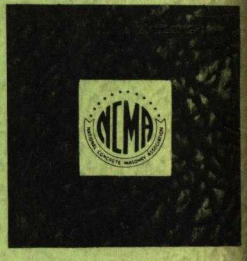
II-20

Fig. II-20. Loop ties embedded in the horizontal mortar joints and engaging vertically reinforced cells in both walls are sometimes used to bond intersecting walls.



II-21

Fig. II-21. View of structure in which bond beam reinforcement has been extended past the face of the wall to permit bonding walls which will be built at a later date to the existing walls.



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